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EXPLANATORY HANDBOOK ON THE

B.S. CODE OF PRACTICE FOR REINFORCED CONCRETE

RECOMMENDED BY THE CODES OF PRACTICE COMMITTEE
FORMED UNDER THE AEGIS OF THE MINISTRY OF WORKS

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PREFACE

IN its form and in its substance this Handbook follows the lines of the Handbook on the D.S.I.R. Code of Practice issued in 1934.

A large and steadily-maintained demand over a period of sixteen years has proved the continuing value of the original Handbook. The present Handbook is a completely revised work based upon the British Standard Code of Practice for the Structural Use of Normal Reinforced Concrete in Buildings issued in 1948. It has been written in the light of recent knowledge and experience.

The Code, which is reproduced in its entirety, is insetted and indicated by black lines in the left-hand margin, and the authors' comments follow the clauses.

The tables and figures which do not appear in the Code itself have been designated Table 1A, 2A, etc., and Fig. 1A, 2A, etc., to distinguish them from the tables appearing in the Code.

We deeply regret the loss of our colleague, W. L. Scott, who was with us until the book was almost complete.

W. H. GLANVILLE.
F. G. THOMAS.

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This Code of Practice represents a standard of good practice and therefore takes the form of recommendations. Compliance with it does not confer immunity from relevant legal requirements, including byelaws.

SECTION 1.—GENERAL

101. Scope. This code deals with the structural use of normal reinforced concrete in buildings. It represents good present-day practice ; future developments or research will be dealt with as occasion may arise, in subsequent editions. Pre-stressed concrete, water-retaining structures and other reinforced concrete constructions of a specialised character, are not included.

It has been assumed in drafting this code that the design of reinforced concrete is entrusted to chartered structural or civil engineers, qualified in reinforced concrete, for whose guidance it has been prepared, and that the execution of the reinforced concrete work is carried out under the direction of a qualified supervisor.

Reference should also be made to Code 114.100 (1950), 'Concrete floors and roofs' and its sub-codes (114.101 to 114.105).

102. Definitions. For the purpose of this code the following definitions apply :

Column head.	An enlargement at the top of a column supporting a flat slab, designed and constructed to act monolithically with the column and with the flat slab.
Column strip.	A portion of a flat slab panel of total width usually one-half the panel width occupying the two areas outside the middle strip.
Columns, long.	Columns having a ratio of effective column length as given in Table 12, clause 312 (*a*), to least lateral dimension greater than 15.
Columns, short.	Columns having a ratio of effective column length as given in Table 12, clause 312 (*a*), to least lateral dimension not exceeding 15.
Core of helically reinforced column.	The portion of the concrete enclosed within the centre line of the helical reinforcement.
Deformed bar.	A bar in which the bond strength exceeds that of a plain round bar by 10 per cent or more.
Diagonal band.	A band of reinforcing bars parallel with a diagonal of a flat slab panel which is reinforced in four directions (i.e. edgewise and diagonally).
Direct band.	A band of reinforcing bars parallel with an edge of a flat slab panel reinforced in four directions (i.e. edgewise and diagonally).
Drop.	The portion of a flat slab above and immediately surrounding the column head and of greater thickness than the remainder of the flat slab panel.

and total sulphur calculated as sulphuric anhydride, are the same as in B.S. No. 12 : 1947 (see under " B.S. for Portland Cement ").

B.S. FOR HIGH-ALUMINA CEMENT, No. 915 : 1947.

Chemical Composition.—The total alumina content shall be not less than 32 per cent. by weight of the whole. The ratio of the percentage by weight of alumina (Al_2O_3) to the percentage by weight of lime (CaO) shall not be less than 0·85 or more than 1·3.

TABLE 1A.—PHYSICAL REQUIREMENTS OF BRITISH STANDARDS FOR CEMENTS.

	B.S. No. 12 (1947)		B.S. No. 146 (1947)	B.S. No. 915 (1947)
	Ordinary Portland cement	Rapid-hardening Portland cement	Portland-blast-furnace cement	High-alumina cement
Fineness. Amount retained on B.S. Test Sieve No. 170 . .	Not more than 10 per cent.	Not more than 5 per cent.	Not more than 10 per cent.	Not more than 8 per cent.
Specific Surface. Sq. cm. per gramme	2250	3250	2250	2250
**Setting time.* Initial set . .	Not less than 30 minutes	Not less than 30 minutes	Not less than 30 minutes	Not less than 2 hours nor more than 6 hours
Final set . . .	Not more than 10 hours	Not more than 10 hours	Not more than 10 hours	Not more than 2 hours after initial set
Soundness. Expansion after 1 hour's boiling	Not more than 10 mm.	Not more than 10 mm.	Not more than 10 mm.	Not more than 1 mm.
Tensile strength (lb. per sq. in.) (1 : 3 mortar briquettes)				
1 day	—	Not less than 300	—	—
3 days	Not less than 300	Not less than 450	Not less than 300	—
7 days	Not less than 375	—	Not less than 375	—
†*Compressive strength* (lb. per sq. in.) (1 : 3 mortar cubes)				
1 day	—	Not less than 1600	—	Not less than 6000
3 days	Not less than 1600	Not less than 3500	Not less than 1600	Not less than 7000
7 days	Not less than 2500	—	Not less than 2500	—

* Normal-setting cements. † This test is optional, except in the ease of high-alumina cement.

The strength test required in B.S. 12 and B.S. 146 is a tensile test on 1 : 3 mortar briquettes, while a compression test on 1 : 3 mortar cubes is optional. In B.S. 915 for high-alumina cement, the compression test only is specified; for this cement, a 2·78-in. cube (of area 50 sq. cm.) must comprise 190 gr. of cement, 570 gr. of sand, and 76 gr. of water. For the other kinds of cement, the quantities are 185 gr., 555 gr., and 74 gr. respectively. In all cases the water content is 10 per cent. by weight of the cement and dry sand.

Although the tensile test may form a useful means of ensuring a satisfactory cement for general purposes, research has shown that it does not provide a measure of the value of cement in making concrete (see Building Research Board Annual Report, 1930, p. 53). While it is clearly best to test the concrete a fairly reliable indication of concrete strength may be obtained from a compression test of 1 : 3 mortar made with standard sand and using a water content of 12½ per cent. of the weight of the cement and sand. Under properly controlled testing conditions the strength of such a mortar should be within ± 10 per cent. of the strength of 1 : 2 : 4 (by weight) concrete made with sound river sand and gravel aggregate and with a water-cement ratio of 0·60 by weight. This test indicates the concrete-making properties of a cement, but must not be regarded as replacing the preliminary tests required by Clause 208 (*a*).

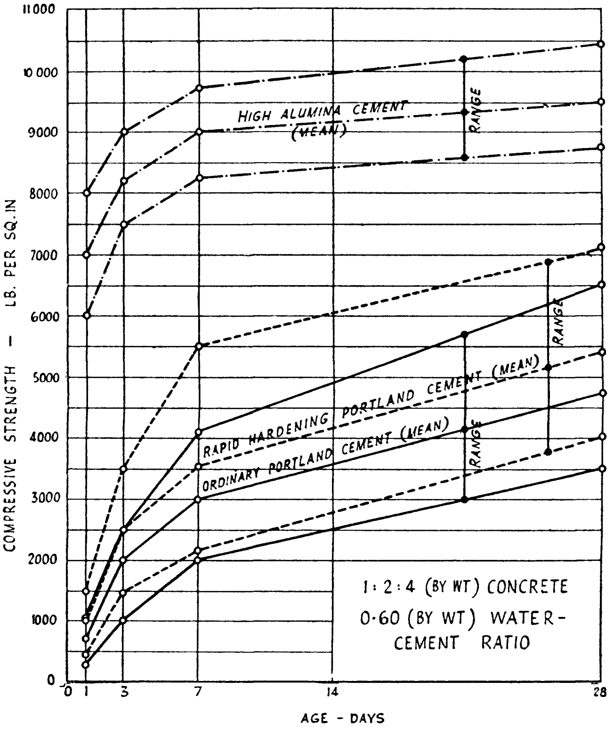

FIG. 1A.—AGE-STRENGTH CURVES FOR CONCRETE.

An indication of the strengths obtained with the three main types of concretes mixed in the proportions of 1 : 2 : 4 is given in *Fig.* 1A. The tests were all made using a water-cement ratio of 0·60 by weight and at a temperature of 64 deg. F. The figure shows the limits between which the strengths of the various grades of cements have fallen for a large number of recent tests made at the Building Research Station.

The effect of the water-cement ratio on the strength of ordinary and rapid-hardening Portland cement concrete is shown in *Table* 2A. The cements are of average quality.

TABLE 2A.—STRENGTHS OF CONCRETE WITH VARIOUS WATER-CEMENT RATIOS.

Water-cement ratio (By weight)	Concrete crushing strength (lb. per sq. in.)				
	Ordinary Portland cement				
	3 days	7 days	28 days	3 months	1 year
0·35	4800	6200	8400	10,200	11,000
0·45	3500	4700	6800	8400	9300
0·60	2000	3000	4700	6200	7000
0·75	1200	2000	3300	4400	5000
	Rapid-hardening Portland cement				
	3 days	7 days	28 days	3 months	1 year
0·35	5500	7000	9200	10,400	11,000
0·45	4100	5400	7500	8600	9300
0·60	2500	3600	5400	6400	7000
0·75	1600	2500	3800	4600	5000

Ordinary Portland and rapid-hardening Portland cements are very similar in character, the latter usually differing from the former only in that it may be more finely ground and slightly altered in chemical composition.

Portland-blastfurnace cement is manufactured in Britain principally in Scotland and is seldom used south of the Border. It is made from a mixture of Portland cement clinker and certain types of slag which, after subjection to a process of very rapid cooling, known as granulation, possess cementitious properties. The clinker and the slag are ground together to give the finished cement. It is usually lighter in colour than Portland cement. The properties of Portland-blastfurnace cement are in general similar to those of Portland cement.

High-alumina cement differs radically from Portland cements both in properties and composition. It is made by completely fusing a mixture of limestone and bauxite. The finished cement has a much lower lime and silica content, and much higher alumina and iron-oxide content, than Portland cement. It is very dark in colour.

Typical compositions of various cements are shown in *Table* 3A.

Apart from differences in physical properties, these cements differ much in their resistance to attack by various agents.

For sea-water work it is necessary that concrete should be as dense as possible to resist the chemical attack of the salts present in the sea, and to prevent the water penetrating the concrete and corroding the reinforcement. Portland-blastfurnace cement is usually considered to be somewhat more resistant

TABLE 3A.—TYPICAL COMPOSITIONS OF CEMENTS.

	Percentage compositions of cements			
	Ordinary Portland cement	Rapid-hardening Portland cement	Portland-blast-furnace cement	High-alumina cement
Lime (CaO)	64	65	57	40
Silica (SiO_2)	20	20	24	5
Alumina (Al_2O_3)	7	6	8	40
Iron oxide (Fe_2O_3)	3	2·5	3	12
Sulphur trioxide (SO_3)	2	2·5	2	None

to the action of sea-water than ordinary Portland cement. The evidence available shows that, when properly made, high-alumina cement concretes are very resistant to sea-water.

Certain soils and ground waters are harmful to concrete; for example, in some soils and clays a considerable amount of gypsum (calcium sulphate) or other sulphates is present, and these, under the worst conditions, can completely disintegrate even the best Portland cement concretes. Hence Portland cement concretes to be laid in direct contact with such types of soils, or waters emanating from them, require special protection, and in certain cases should not be used. As a rough guide it may be assumed that if a water contains more than about 0·5 per cent. of sulphate salts, or a soil more than about 0·25 per cent. of sulphur trioxide (SO_3) soluble in dilute acids, special consideration should be given before using Portland cement concrete. High-alumina cement concrete if well made is resistant to soils and waters of high sulphate content, while Portland-blastfurnace cement is usually regarded as rather more resistant than Portland cement.

Some waters from high moorland areas are distinctly acid and attack concretes, though their action is not so serious as that of sulphate-bearing soils and waters. The attack tends to take the form of surface solution of the cement and exposure of the aggregate. High-alumina cement seems to be rather more resistant than Portland cement under such conditions.

In industrial uses of concrete many liquids, oils, and fats are encountered which affect concrete. Many of these present special problems, and special treatment appropriate to the case is required. Consideration should therefore be given to the adequate protection of concrete in all the various cases suggested later, and advice sought where necessary. Milk and whey are harmful to both Portland and high-alumina cement concretes, and some special surface protection is required; for this purpose a strong impermeable granolithic rendering may be used provided that it is washed down at least daily with a copious supply of water. Animal and vegetable fats such as arise in the soap, fat, and food industries often have a serious effect on concrete. Sugar solution, some mineral salts, all the stronger and some of the weaker inorganic and organic acids, are detrimental to concrete in varying degrees. Where contact with such agents is likely to occur, concrete should be protected by special surface treatment or by a suitable insulating material. These are matters on which specialist advice should be sought.

202. Aggregates. The aggregates should be either :—

(*a*) Fine and coarse aggregates complying with the requirements of B.S. 882, 'Concrete aggregates and building sands' or

(*b*) such other types of aggregate as may be suitable, having regard to their strength, durability and freedom from harmful properties.

In the case of subsidiary members, which include wall panels and precast floor and roof slabs, the following aggregates also may be used :

(*c*) Air-cooled blastfurnace slag (coarse aggregate) complying with the requirements of B.S. 1047, 'Air-cooled blastfurnace slag (coarse aggregate).'

(*d*) Foamed blastfurnace slag complying with the requirements of B.S. 877, 'Foam blastfurnace slag for concrete aggregate.'

(*e*) Crushed clay brick and tile containing not more than 1 per cent of sulphate. As there is no British Standard at present it should be noted that the porosity of clay brick and tile varies considerably and preference should be given to the densest type of these materials.

(*f*) Natural pumice free from clay, mud, dust and other impurities.

The grading of the aggregates should be such as to produce a concrete of the specified proportions which will work readily into position without segregation and without the use of an excessive water content. The grading should be carefully controlled throughout the work so that it conforms closely to that used for the preliminary tests.

203. Maximum size of coarse aggregates. The maximum size of coarse aggregate should be as large as possible, within the limit specified in the appropriate British Standard as given in clause 202, but in no case greater than one-quarter of the minimum thickness of the member, provided that the concrete can be placed without difficulty so as to surround all reinforcement thoroughly and to fill the corners of the formwork.

For heavily reinforced concrete members, e.g. the ribs of main beams, the nominal maximum size of *the* aggregate *should usually be restricted to ¼ in.* less than the minimum lateral distance between the main bars, or ¼ in. less than the minimum cover to the reinforcement, whichever is the less.

Where, however, as in solid slabs, the reinforcement is widely spaced, *limitation of the size of* aggregate is *not so important and the nominal maximum* size may sometimes be as great as, or greater than, *the* minimum cover, except where porous aggregates are used.

For general purposes a maximum size of ¾ in. is usually satisfactory.

For most reinforced concrete work the aggregates consist of naturally-occurring materials, crushed or uncrushed, and B.S. 882 specifies the requirements for the quality and grading of these aggregates. Fine aggregate is defined as aggregate mainly passing a $\frac{3}{16}$-in. sieve and coarse aggregate as aggregate mainly retained on this sieve, the tolerance implied by the word " mainly " being specified for each size and class of aggregate. The aggregates must be hard, strong, durable, clean, and free from adhering coatings.

Inadequate washing frequently leaves aggregate containing clay with clay films over the surface of the particles. It is particularly important that this should be guarded against as it prevents adhesion of the cement to the aggregate and results in a weak concrete. Similarly, dust produced during the process of crushing may be left adhering to the aggregates in quantities sufficient to be detrimental.

Harmful Impurities and Materials to be Avoided.—B.S. 882 specifies that the aggregate shall contain no harmful material in sufficient quantity to affect adversely the strength or durability of the concrete or to attack the reinforcement, and defines tests for ascertaining whether the aggregates contain excessive amounts of clay, silt, or fine dust, or excessive organic impurities.

Although some aggregates may be perfectly satisfactory for certain classes of plain concrete work, their use for reinforced concrete is not always desirable. Examples of these are coal-residue products which are used successfully in partition slabs of plain concrete, but may lead to serious deterioration of reinforced concrete.

Clinker or breeze is the fused or sintered ashes remaining from the combustion of coal. Clinker and breeze aggregates vary from hard well-fused clinker containing little unburnt matter to rather poorly sintered materials containing a considerable proportion of partially-burnt, and some unburnt, coal. The hard-burnt clinker gives a heavier and denser concrete than the more poorly sintered materials. Serious troubles are sometimes encountered in the use of clinker and breeze aggregates. They arise from the presence of certain types of unburnt or slightly-burnt coal in the aggregate producing expansion in the set concrete, which under some conditions may continue for a long period. Only certain types of coal possess this dangerous property. If clinker or breeze aggregates are to be used for any purpose care should be taken to ensure that they are sound. B.S. 1165 sets out requirements for clinker which ensure that material conforming to this specification will give satisfactory durability in *plain* concrete.

Coke-breeze consisting only of small coke from gasworks should not suffer from unsoundness due to the presence of unburnt coal, but as a rule its physical properties are such that it can make only a rather poor, though light, concrete. Coke-breeze should contain over 75 per cent. of combustible coke matter, and less than 25 per cent. of ash.

Clinker, breeze, or coke-breeze concretes are unsuitable for use in contact with any reinforcement or steel. They are as a rule rather permeable, and tend to maintain themselves in a moister condition than their surroundings. These factors, together with the influence of the sulphur compounds contained, usually result in corrosion of any steel in contact with them.

Copper slag, forge breeze, dross, and similar materials, have doubtful properties as concrete aggregates, and they should not be used.

Gypsum and materials containing much soluble sulphate should never be used as aggregates in any concrete, owing to their very harmful effect on set cement. For example, alabaster is a form of calcium sulphate, and if used as a concrete aggregate will inevitably lead to failure.

Care is also required in the use of dolerites as aggregates in concrete. Many of these form quite satisfactory aggregates, but certain of them contain mineral constituents which produce expansion in the hardened concrete. In general, the fine material in such an aggregate is much more likely to be dangerous than the coarse material. The term whinstone is often used commercially for dolerite, but it also covers many other rocks. Unfortunately, satisfactory tests for detecting unsuitable material are not yet available. Where, however, an aggregate has been proved satisfactory by the existence of good concrete at least several years old, it may reasonably be considered suitable for use. Proof of the satisfactory behaviour of the rock when not made into concrete is not adequate.

Although it is wise to regard with suspicion all aggregates that fail to pass the test for organic impurities, some of these aggregates will nevertheless produce sound concrete. For example, failure may in some cases be due to the presence of small particles of oil shale, but this impurity, although causing excessive discoloration in the test, is unlikely appreciably to affect the quality of the concrete. In such cases crushing tests should be made of the concrete, and at the same time comparative tests should be made using other aggregates known to be sound. If the strengths are similar the aggregate may be used with confidence.

Grading.—In the production of good concrete the aim should be to obtain a sufficient degree of workability to ensure a compact dense concrete free from air voids, surface honeycombing, and segregation, with the least water consistent with the amount of tamping practicable. The degree of workability and the amount of water required are largely governed by the grading and shape of the particles of aggregate. Vibratory methods of compacting concrete have been employed with gradings not otherwise suitable, usually with a lower sand content.

It is impossible to indicate any particular grading as ideal, since many variations will produce concrete of the same strength; there are, however, certain important factors which should be carefully noted. It is not necessary that the complete range of particle sizes should be present, although, to reduce segregation during placing, some degree of continuity of grading throughout the range of particle sizes is desirable.

FINE MATERIAL.—The strength and workability of concrete are largely governed by the amount of fine aggregate passing a No. 52 B.S. sieve. Except with concrete rich in cement, lack of such material is usually accompanied by harshness and unworkability, but provided that adequate consolidation is possible the resultant concrete should be of good quality. An excess of fine material, although improving workability, is undesirable because it leads to the use of too much water and, therefore, to low strength. On most work the amount of tamping is limited by practical considerations, and it is therefore necessary to effect a compromise between the grading giving maximum strength and that providing the most workable concrete. For 1 : 2 : 4 concrete it may be taken as a general guide that not more than 30 per cent. and not less than 5 per cent. should pass through a No. 52 B.S. sieve. Concrete in the proportions of 1 : 1½ : 3 and richer is satisfactory without this fine material, the increased cement ensuring adequate workability.

It has been found that typical good sands contain between 30 per cent. and 55 per cent. passing a No. 25 B.S. sieve, 50 per cent. to 70 per cent. passing a No. 14 B.S. sieve, and 75 per cent. to 90 per cent. passing a No. 7 B.S. sieve. It does not follow, however, that all satisfactory sands are within these limits.

INTERMEDIATE SIZES.—Harshness and lack of workability or bad surface finish may result from an excess of particles of intermediate sizes. With a ¾-in. maximum-size coarse aggregate trouble may arise from an excess of particles ranging in size between the No. 7 B.S. sieve and the $\frac{3}{16}$-in. B.S. sieve. Ordinarily this quantity should not exceed about 20 per cent. of the fine aggregate. With aggregates of larger maximum size the troublesome intermediate sizes may be larger, even up to ½ in.

SHAPE OF PARTICLES.—Two similarly graded and equally clean aggregates, one consisting of rounded and the other of angular or crushed particles, will exhibit markedly different degrees of workability when mixed with equal quantities of water and cement, but, when sufficiently compacted, should have identical compressive strengths. The rounded aggregate will require less water to bring it to a condition suitable for deposition and will produce a more workable concrete than one composed of angular pieces.

(See also comment on Clause 206 for notes on combined gradings.)

204. Water. The water should be clean and free from harmful matter.

Harmful materials in water are generally of an organic nature; examples of unsuitable water are found in peaty districts. Where efflorescence is not objected to there is little harm in using sea-water for mixing concrete.

205. Reinforcement. The reinforcement should be:

(*a*) Rolled steel bars or hard drawn steel wire, complying with the requirements of B.S. 785 'Rolled steel bars and hard drawn steel wire for concrete reinforcement.'

(*b*) Cold twisted steel bars complying with the requirements of B.S. 1144 'Cold twisted steel bars for concrete reinforcement.'

(*c*) Steel fabric complying with the requirements of B.S. 1221, 'Steel fabric for concrete reinforcement.'

(*d*) Such other reinforcement as may be suitable, having regard to the yield-point stress, ductility, ultimate resistance to tension and other essential properties of the completed reinforcement as produced in readiness for use in the reinforced concrete.

The ultimate load-carrying capacity of a reinforced concrete member is controlled, so far as the contribution of the reinforcement is concerned, by the load at which the yield point of the steel occurs. It is therefore important that the yield-point stress should be considered in relation to the working stress in order to determine the load factor for the reinforced concrete. Except in the case of mild steel bars, a minimum yield stress is guaranteed by the manufacturers in accordance with the requirements of the appropriate British Standard. For mild steel, it is apparently assumed in Clause 303 (*b*) that the yield stress is at least 36,000 lb. per square inch.

206. Concrete proportions. The fine aggregate and the coarse aggregate should be measured separately.

Except where high alumina cement is used, the proportions of cement to fine aggregate and coarse aggregate should be as set out in Table 1 or in any intermediate proportions in which the volume of coarse aggregate is twice the volume of fine aggregate. Where, however, a denser or more workable concrete *can be produced by a variation in the ratio of the volume of coarse aggregate to that of the fine aggregate, this ratio may be varied within the limits 1½ to 1* and 2 to 1. The requirements of clause 207 should be based on the ratio of the sum of the volumes of fine and coarse aggregates, each measured separately, to the quantity of cement.

Concrete made with high alumina cement should be mixed in the proportions of 7½ cubic feet of total fine and coarse aggregates, measured separately, to 112 lb. of cement. The ratio of the volume of the coarse aggregate to that of the fine aggregate should be between 1½ to 1, and 2 to 1.

In proportioning concrete, the quantity of cement should be determined by weight. The quantities of fine and coarse aggregates should be determined either by volume or, preferably, by weight. In the latter case the weight should be determined from the volume required by the table and the density of the aggregate. The proportions given above in this clause and in Tables 1 and 2, are based on the assumption that the aggregates are dry. If aggregates are moist, due allowance must be made for bulking. See clause 805.

The quantity of water used for reinforced concrete should be sufficient, but not more than sufficient, to produce a dense concrete of adequate workability for its purpose, which will surround and properly grip all the reinforcement. So far as possible, the consistence of the concrete should be controlled by maintaining a water/cement ratio that is found to give a concrete which is just sufficiently wet to be placed and compacted without difficulty, with the means *available*. In any portion of a member where the reinforcement is congested, a small increase in the water/cement ratio will often lead to a more efficient embedment of the bars, which will outweigh the reduction in basic strength of the concrete due to the higher water content. On the other hand, the amount of water can usefully be reduced when vibratory methods are adopted for compacting the concrete ; in such cases the use of a concrete of wet consistence is both unnecessary and undesirable and may lead to segregation.

For concrete with uncrushed gravel aggregates, the water/cement ratio (by weight) should not exceed 0·43, 0·51 and 0·58* for 1 : 1 : 2, 1 : 1½ : 3 and 1 : 2 : 4 nominal mixes, respectively ; and with compacting by vibration, these values should normally be reduced by at least 20 per cent. For other aggregates the water/cement ratio may need to be slightly greater. The water/cement ratio should not be allowed to exceed the ratio used in the preliminary tests by more than 10 per cent.

The consistence should be controlled by direct measurement of water content, making allowance for any water in the fine and coarse aggregates, and the 'slump' test described in clause 801 may be used to give an approximate guide.

Grading of Combined Aggregates.—Since in all cases the maximum size of fine aggregate is limited essentially to 3/16 in., while the maximum size of coarse aggregate may be anything from, say, ⅜ in. upwards, it is clear that the best proportions for the fine and coarse aggregate will not remain constant, and that the best grading of fine aggregate for one maximum size of coarse aggregate will probably be quite unsuitable for another. It can be assumed as a rough guide that from 45 per cent. to 75 per cent. of the total aggregate (fine and coarse) should pass through a sieve of aperture size equal to one-half of the maximum size of the coarse aggregate.

Research shows that aggregates (¾-in. maximum size) graded between the

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* These values correspond respectively to 4¾, 5¾ and 6½ gallons of water to each 112 lb. of cement.

following limits are consistently satisfactory. With some materials it will be found impracticable to proportion fine to coarse aggregate so as to bring the combined grading within these limits, but it is nevertheless desirable to do so as closely as possible.

Sieve size	Percentage passing (by weight)
¾-in. mesh	100
⅜-in. mesh	55–65
3/16-in. mesh	35–42
No. 7	28–35
No. 14	21–28
No. 25	14–21
No. 52	3–5
No. 100	0–3

As the maximum size of coarse aggregate becomes smaller it will usually be necessary to reduce the proportion of coarse to fine aggregate. For example, if the proportions for a ¾-in. maximum size are 1 : 2 : 4 the content of coarse aggregate will need to be reduced for ⅜-in. maximum size, and the proportions 1 : 2 : 3 will probably be suitable and will give practically the same strength. With concrete rich in cement the modifications required in the proportions are *less than with leaner mixes ; in fact, in the* case of a 1 : 1 : 2 *concrete it is normally* unnecessary to reduce the quantity of coarse aggregate. For 1½-in. maximum size the use of the proportions 1 : 2 : 4 will give concrete of strength not less than with ¾-in. maximum size, but provided that the coarse aggregate is reasonably graded the fine aggregate content may with advantage be reduced to give proportions of 1 : 1½ : 4½. (Note, however, that the Code does not permit a coarser combined grading than that possible with a 1 : 2 : 4 concrete.)

Where it may be desired to use a very fluid concrete, for example where the reinforcement is congested, it is advisable to increase slightly the content of sand. This increase in sand does not (with fluid concrete) necessitate more water ; in fact, such concrete can be placed with a lower water-cement ratio than concrete with less sand. For example, if a 6-in. slump is essential it would in normal circumstances be better to use a 1 : 2½ : 3½ concrete instead of 1 : 2 : 4.

Quantities are given in *Table* 4A for approximately one cubic yard of finished concrete for both gravel and broken stone aggregate. Amended proportions and quantities based on the ratios previously given are also included for ⅜-in. and 1½-in. maximum size aggregate. These figures are based on typical materials as used in practice, except that no allowance has been made for bulking of sand due to moisture.

Concrete should be proportioned so as to ensure both adequate workability and sufficient strength, taking into account the variability likely to occur and the interrelationship of the effects of aggregate grading, water content, and cement content. Useful information on the subject is given in a Road Research Laboratory publication, Road Note No. 4, " Design of Concrete Mixes."

When the best grading has been determined and made the basis of the preliminary strength tests, it is desirable to control the grading throughout the work within reasonable limits. In general, provided that the ratio of water to cement is not varied and the concrete is efficiently compacted, large variations

207. Strength requirements for concrete.

(*a*) WHEN AGGREGATES COMPLY WITH B.S. 882.

(i) *Portland cement concrete or Portland-blastfurnace cement concrete.* Concrete made with Portland cement or Portland-blastfurnace cement should comply with the strength requirements of Table 1, works test, Cols. 3 and 4. Where intermediate proportions of aggregate to cement are used, as provided in clause 206, the minimum crushing strengths required should be obtained by proportion from the two nearest nominal mixes.

TABLE 1.—PROPORTIONS AND STRENGTH REQUIREMENTS FOR PORTLAND CEMENT CONCRETE AND PORTLAND-BLASTFURNACE CEMENT CONCRETE WITH AGGREGATES COMPLYING WITH B.S. 882

1	2		3		4	
Nominal mix	Cubic feet of aggregate per 112 lb. of cement		Minimum resistance to crushing (cube strength) in lb./sq. in. within 28 days after mixing		Alternative minimum resistance to crushing (cube strength) in lb./sq. in. within 7 days after mixing	
	Fine	Coarse	Preliminary test	Works test	Preliminary test	Works test
1 : 1 : 2	1¼	2½	5250	4500	3420	3000
1 : 1½ : 3	1⅞	3¾	4375	3750	2840	2500
1 : 2 : 4	2½	5	3500	3000	2275	2000

Where difficulty is found in reaching the requisite cube strengths in Table 1, figures 25 per cent lower than those given may be accepted, provided that the working stresses are reduced in the same ratio.

(ii) *High-alumina cement concrete.* High-alumina cement concrete should comply with the strength requirements of Table 2. (Works test.)

TABLE 2.—PROPORTIONS AND STRENGTH REQUIREMENTS FOR HIGH-ALUMINA CEMENT CONCRETE WITH AGGREGATES COMPLYING WITH B.S. 882

Nominal mix	Cubic feet of aggregate per 112 lb. of cement		Minimum resistance to crushing (cube strength) in lb./sq. in. within 2 days after mixing	
	Fine	Coarse	Preliminary test	Works test
1 : 2 : 4	2½	5	6000	5000

Two grades of concrete are permitted by the Code; these will be referred to as Normal Grade and Lower Grade.

The Normal Grade concrete must satisfy the strength requirements given in *Table* 1 or *Table* 2 and this grade should normally be used. With sound aggregates and cement of average quality, there should be no difficulty in producing concrete of this grade.

In practice, it is sometimes impossible or impracticable to obtain materials of the quality desired and the cement, the aggregate, or both, may, although satisfying the minimum requirements of Clauses 201 and 202, be below the average standard. In such cases it may be necessary to adopt the Lower Grade.

It should be noted that the concession in regard to reduced strengths is not accompanied by any modification to the general requirements of the Code in connection with the supervision or workmanship. The concrete work must be properly controlled and efficiently supervised whatever grade of concrete is adopted, and the Lower Grade is introduced only to make allowance for the possibility that the materials available may be below average in quality.

The strength of Portland cement concrete may be checked at seven days or 28 days, or at both ages. The quality may be assumed to be satisfactory if the strength at seven days (Works Test) complies with the requirements specified in Clause 207 (*a*) for this age, without the necessity for further tests at 28 days. If the strength at seven days is not sufficient, however, the quality of the concrete may still be considered adequate if the strength in subsequent tests at 28 days attains the value specified for this age. Reference to the table of concrete strengths (*Table* 2A) indicates that the ratio of strength at seven days to that at 28 days varies, in the typical cases given, between 0·61 and 0·74 for ordinary Portland cement concrete and between 0·66 and 0·76 for rapid-hardening Portland cement concrete, the value being greater for the lower water contents. The ratio of the minimum strengths at seven days specified in the Code to those at 28 days is 0·67. With a dry mix incorporating rapid-hardening Portland cement it would be possible occasionally to comply with the strength requirements of the Code on the basis of the strength at seven days, even though the strength at 28 days was up to 10 per cent. less than the strengths specified for this age.

Apart from this possible slight advantage, rapid-hardening Portland cements are given no preferential treatment in the Code. Such cements are therefore to be regarded as of value insofar as they facilitate speedier construction (see page 111) and not as ultimately producing a sounder or stronger structure than ordinary Portland cements.

Where the proportions of aggregate to cement adopted are intermediate between those specified in *Table* 1, the strength requirements must be modified accordingly. If the two nearest defined mixes are such that the sums of the volumes of the coarse and fine aggregates per 112 lb. of cement are S_1 and S_2 respectively, and the corresponding minimum concrete strengths required are u_1 and u_2 (for the particular type and age of test considered), then for an intermediate mix S' the corresponding strength required will be

$$u' = u_2 + \frac{(u_1 - u_2)(S_2 - S')}{S_2 - S_1}$$

EXAMPLE.—For Normal grade concrete mixed in the proportions of 112 lb. of cement to 2 cu. ft. of fine aggregate to 3 cu. ft. of coarse aggregate, the sum of the volumes of the aggregates is 5 cu. ft. This lies between the 1 : 1 : 2 and 1 : 1½ : 3 nominal mixes, for which the sums of the volumes of the aggregates representative of only comparatively small masses of concrete. In general, the heat generated by the cement in setting and hardening will result in increased temperatures, and therefore enhanced rates of hardening for larger masses of concrete.

The frequency with which samples of concrete should be taken for strength tests is not specified, and clearly depends on the type of construction and the volume of similar concrete placed. It is recommended that as a minimum six cubes of each type of concrete be made weekly, three for test at seven days and three for test at 28 days. It should be noted from Clause 802 (*e*) that the *average* strength from the three cubes tested at a particular age may be employed as a criterion of the compressive strength of the concrete so long as the difference between the maximum and minimum strengths of the three specimens does not exceed 15 per cent. of the average strength.

Transverse Works Strength Tests.—The transverse strength test described in Clause 804 may be used as a convenient method of obtaining an early guide to the quality of the concrete. *Table* 3 sets out strengths which might reasonably be expected, but are not demanded by the Code. Preliminary tests should be made to determine more closely the modulus of rupture corresponding to the materials and concrete mix used.

It is possible to obtain more information from the test specimens than the transverse strength only, since the two parts of the specimen after it has been broken can be used to obtain an approximate value for the compressive strength. This is done by loading each half beam through steel plates 4 in. square as shown in *Fig.* 2A. The local bearing stress under these plates at crushing of the concrete is usually only 3 per cent. or 4 per cent. greater than the cube strength of the concrete, and hence the test is described as the " equivalent cube " test.

The transverse strength is normally more variable than the compressive strength, and at least three specimens are desirable to check the strength at a particular age. The six halves can then be used for two sets of three " equivalent cube " tests. A convenient and useful arrangement may be to make the transverse tests at three days as an early check of the quality of the concrete, and the " equivalent cube " tests at seven days and 28 days to supplement the data from the works-cube strength tests. The relationship between the cube strengths and the " equivalent cube " strengths can thus be established for the particular concrete, and it may be reasonable to reduce the number of regular cube tests when " equivalent cube " test results are available.

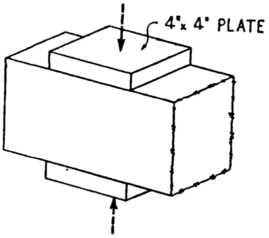


FIG. 2A.—" EQUIVALENT CUBE " TEST ON HALF OF A 16-IN. BY 4-IN. BY 4-IN. PRISM AFTER TRANSVERSE TEST.

SECTION 3.—DESIGN CONSIDERATIONS

301. Basic assumptions. Computation of stresses should accord with the laws of mechanics and the recognised general principles relating to the design of reinforced concrete. It may be assumed :—

(*a*) that both steel and concrete are elastic within the range of the permissible stresses and that the modular ratio, m, is equal to 15,

(*b*) that plane sections remain plane, and

(*c*) that all tensile stresses are taken by the reinforcement, except that the concrete may be assumed to resist diagonal tension within the limits of shear stress specified for plain concrete in clause 303 (*a*).

The assumptions made are all well known and are commonly accepted as a basis for the design of members subjected to bending. In varying degrees all of them are inaccurate, and it is known that their use does not lead to a true conception of the stresses actually developed. They should be regarded as a framework which, when used properly in conjunction with the permissible stresses, enables designs, the safety of which has been proved by practical experience and tests, to be made in a simple manner and with comparative ease.

Although the assumption that steel is perfectly elastic may be regarded as true within the range of permissible stresses, it should be realised that the relation of stress to strain for concrete is dependent on the time the concrete has been under load. For example, an instantaneous stress, within the permissible limits, when induced in concrete will result in a strain which for all practical purposes is proportional to the stress. If this stress is induced and removed instantaneously the recovery of the concrete to its initial length will be complete and for these theoretical conditions there will be no permanent set. If, however, the stress is not instantaneous the concrete will have an additional deformation which, although still proportional to the applied stress, will not be recovered on removal of the load. This additional deformation is known as creep.

Creep takes place at a rate decreasing with time in all concrete maintained under load, and it may tend to a limiting value four or five times as great as the purely elastic movement. Moreover, immediately prior to failure an additional amount of creep takes place. It is because of creep that it is safe to use in the framework of assumptions values of the modular ratios that are higher than the true or instantaneous values.

Since the creep occurring in concrete is proportional to the stress present it is possible to make an approximate estimate of the stresses developed in a member after it has been loaded for a time, by using in the calculations a higher value of the modular ratio than the true one. This value has been termed the " effective " modular ratio. Typical values are shown in *Fig.* 3A, in which they are related to the cube strength of the concrete. It will be seen that the effective

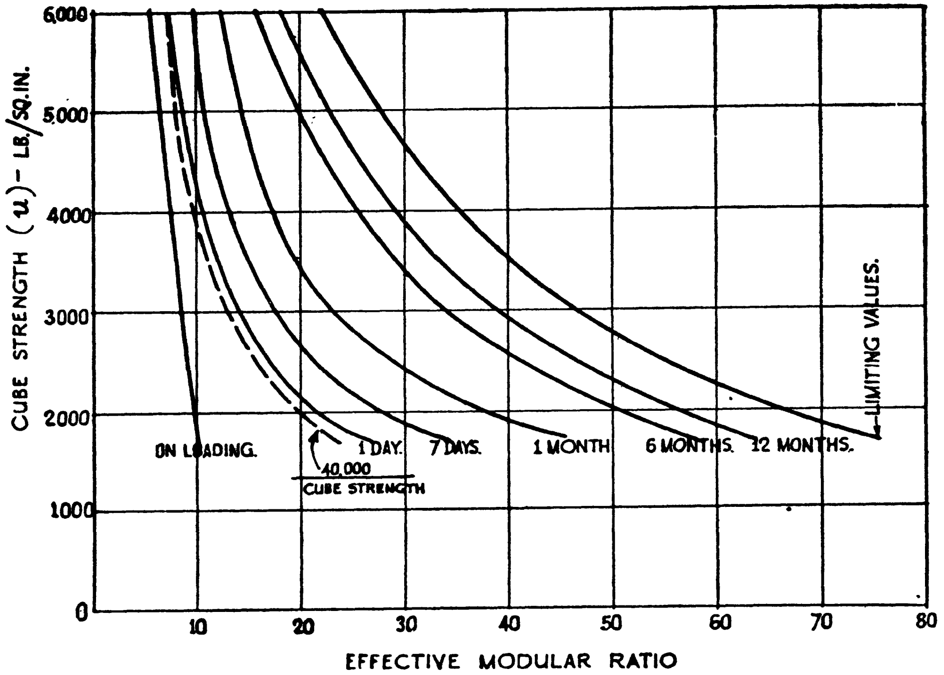


FIG. 3A.—EFFECTIVE MODULAR RATIOS AFTER VARYING PERIODS UNDER LOAD.

modular ratios decrease as the strength of the concrete increases, and, in order to maintain a uniform standard of safety, it is preferable to vary the modular ratio. Values equal to 40,000 divided by the works cube strengths (in lb. per square inch) are satisfactory for concrete strengths within the limits given in *Tables* 1 and 2. The curve representing these values is shown in *Fig.* 3A. It approximates fairly closely to the curve for the effective modular ratios after one day under load.

Although the clause specifies that the modular ratio may be assumed to be 15 apparently for all types of concrete, other values may presumably be adopted where justifiable. In the case of low-strength concrete, permitted by Clause 207 (*b*), the modular ratio can usually be assumed to be higher than 15, but experimental data would be necessary to establish a suitable value.

The effect of creep is always to reduce the stress in the concrete and to increase the stress in the steel. Although the stresses at the time of loading may be greater than those computed for the concrete and less than those computed for the steel, the condition will ultimately be reversed. The stress in the concrete may then be less than one-half of its initial value, while the tension in the steel in sections subjected to bending may be 15 per cent. higher than the computed value. The compressive stress in steel may be increased to several times the initial value, but this is not a matter for concern since it is compensated by a decrease in concrete stress, and a redistribution of stress occurs near the failing load. The factor of safety against ultimate failure remains practically unaltered. (See Building Research Technical Paper No. 12, by W. H. Glanville.)

The assumption that concrete resists no tensile forces becomes less accurate as the quantity of reinforcement is reduced. With very low percentages of reinforcement the whole of the concrete section may be stressed, but as a basis for calculation the assumption is on the safe side.

302. Loadings. The loadings should be in accordance with the Code of Functional Requirements of Buildings, Chapter V, 'Loading.'

For the purpose of calculating dead loading, the weights of materials should, unless otherwise agreed, be taken to be as set forth in B.S. 648, 'Schedule of unit weights of building materials.'

For ordinary construction the weight of reinforced concrete may be taken as 150 lb. per cubic foot, but, when the percentage of steel exceeds 2 per cent, some greater weight may be more appropriate. Where light weight aggregates are used a smaller appropriate weight may be taken.

The British Standard Schedule of Weights of Building Materials B.S. 648 sets out " standard " weights which represent fair average values that may be regarded as sufficiently accurate for general purposes in calculating dead loads in buildings. A guide to the range of weight is given in an appendix to B.S. 648.

So far as concrete is concerned, the " standard " weights and the probable range are as given in the following table.

TABLE 5A.—WEIGHTS OF CONCRETE AND REINFORCED CONCRETE.

	Aggregate	" Standard " weight (lb. per cu. ft.)	Range (lb. per cu. ft.)
Plain concrete	Sand or gravel or crushed natural stone (B.S. 882)	144	140–150
	Foamed slag (B.S. 877)	80	60–115
	Brick	115	110–135
	Pumice	70	35–70
Reinforced concrete	Sand or gravel or crushed natural stone, with :		
	1 per cent. steel	—	144–154
	2 do.	150	148–158
	5 do.	—	160–170
	10 do.	—	180–190

TABLE 6.—PERMISSIBLE STRESSES FOR CONCRETE WHERE AGGREGATES DO NOT COMPLY WITH B.S. 882.

Minimum resistance to crushing (cube strength) in lb./sq. in. within 28 days after mixing		Permissible concrete stresses in lb./sq. in.				
		Compression		Shear	Bond	
Preliminary test	Works test	Direct	Due to bending		Average B_1, see clause 307 (*a*)	Local B_2, see clause 307 (*b*)
u_p		$\frac{u_p}{8}$	$\frac{u_p}{6}$	$\frac{u_p}{80}$	$\frac{u_p}{60}$	$\frac{u_p}{40}$
	u_w	$\frac{3u_w}{16}$	$\frac{u_w}{4}$	$\frac{3u_w}{160}$	$\frac{u_w}{40}$	$\frac{3u_w}{80}$

Where the aggregates do not comply with the requirements of B.S. 882, no minimum strengths are specified in terms of definite proportions in Clause 207 (*b*), and the permissible stresses are given as functions of the actual strengths obtained in the preliminary and works tests. In view of this concession, and of the likelihood of greater variation of concrete strength when such aggregates are used, the factor of safety chosen is higher than that assumed for the figures in *Table* 4. At the same time a greater difference is assumed between the strengths obtained in the preliminary test and the works test, and this leads to a further safeguard since the working stresses must not exceed the lesser of the two values obtained from the preliminary tests and the works tests respectively.

(iii) *Vibrated concrete.* Where all the concrete in a job is thoroughly compacted with an approved mechanical vibrator and a reduced water/cement ratio adopted in the work, the permissible stresses on concrete in compression, in Table 4, may be increased by 10 per cent provided the cube strengths specified in Table 1 are increased also by 10 per cent.

In such cases the test cubes should have the same water/cement ratio as the concrete in the work.

The greatly increased strengths possible with mechanical vibration of the concrete, coupled with a reduced water content, have already been mentioned in connection with Clause 206.

(iv) *Design of slabs and rectangular beams.* Where a slab or rectangular beam is reinforced with 1 per cent or under of mild steel stressed to not more than 18 000 lb./sq. in. or with 0·67 per cent or under of high yield point steel stressed to not more than 27 000 lb./sq. in., no concrete compression stresses need be calculated. The lever arm, in such cases, may be taken as 0·88 of the effective depth.

Reference to *Table* 11A shows that for Normal-grade concrete (with aggregates complying with B.S. 882) the economic percentage is at least 1·26 when the permissible stress in the steel is 18,000 lb. per square inch and at least 0·66 when the permissible stress in the steel is 27,000 lb. per square inch. Clause 303 (*a*) (iv) does not therefore introduce any new concession for such concrete.

For Lower-grade concrete the corresponding economic percentages are only 0·80 and 0·41, and theoretically the adoption of the simple rule in Clause 303 (*a*) (iv) will lead to overstressing of the concrete. It will be remembered, however, that the actual modular ratio for Lower-grade concrete is greater than for Normal-grade, whereas *Table* 11A is based on a constant value of 15. Also, experiments have shown that the percentages of steel corresponding to failure of a beam by simultaneous yielding of the steel and crushing of the concrete are appreciably greater than those deduced from the permissible stresses for steel and concrete given in the Code, as shown in *Table* 11A.

Although not specifically mentioned in the clause, it must be assumed that it applies only to concretes for which definite strength requirements are given in the Code. That is, concrete compressive stresses in beams and slabs may not be neglected if the works test strength at 28 days is less than 2250 lb. per square inch.

(*b*) REINFORCEMENT

(i) *Steel to B.S.* 785 *and B.S.* 1144. The tensile and compressive stresses in steel reinforcement of reinforced concrete, when the steel complies with the requirements of B.S. 785 or B.S. 1144, should not exceed those shown as appropriate for each designation of stress in Table 7.

Steel complying with the above specifications but having a guaranteed yield point higher than those stated may be used with a correspondingly higher stress, but in no case with a stress exceeding those shown in paragraph 303 (*b*) (ii).

(ii) *Tensile stresses in steel not complying with B.S.* 785 *and B.S.* 1144. The tensile stresses in reinforcement should not exceed 0·5 times the guaranteed yield point stress of the completed bar as produced in readiness for use in reinforced concrete. In determining the tensile stress permissible, however, regard should be given to evidence that undesirable cracking of the concrete will not occur and while a stress of 27 000 lb./sq. in. should not at present be exceeded, a lower limit may need to be adopted in circumstances of exposure to corrosive influences.

(iii) *Compression stresses in steel.* Compression stresses in reinforcement in beams or slabs may be calculated either as giving assistance to the concrete or as taking the whole compression. In the first case the steel stress should be 15 times the stress in the concrete at the same distance from the neutral axis. In the second case the stress should not exceed half the guaranteed yield point or 20 000 lb./sq. in. whichever is less.

At present it is recommended that twin twisted and other bent bars should not be allowed for in compression.

The basic factor of safety on the yield-point stress of the steel is taken as 2. Other considerations may, however, decide the limit to which it is safe to stress the steel. These arise from cracking that occurs on the tension side of reinforced concrete members subjected to bending. This cracking must not be allowed to become so extensive as to be unsightly or to lead to corrosion of the steel. In general, the cracking is largely independent of the quality of the

Table 7.—Permissible Stresses in Reinforcement.

Designation of stress in steel reinforcement	Permissible stress in lb./sq. in.				
	Mild steel B.S. 785 1938	Mild steel with guaranteed minimum yield point ; and medium tensile steel to B.S. 785	High tensile steel to B.S. 785	Cold twisted steel to B.S. 1144	
				Single bars	Twin bars
		Guaranteed minimum yield point lb./sq. in.			
		37 000 to 44 000	44 800 to 51 500	60 000	54 000
		depending on the size of the bar			
1 Tension in helical reinforcement in a column.	13 500	15 000	18 000	18 000	18 000
2. Tension other than in helical reinforcement in a column or in shear reinforcement.	18 000	Half guaranteed yield point, but not more than 27 000 except in the case of pre-stressed concrete.			
3. Tension in shear members in beams.	18 000	Half guaranteed yield point, but not more than 20 000.			
4. Compression in column bars other than twin twisted or other bent bars.	18 000	Half guaranteed yield point, but not more than 20 000.			—
5. Compression in bars other than twin twisted or other bent bars, in a beam or slab when the compressive resistance of the concrete is taken into account.	The calculated compressive stress in the surrounding concrete multiplied by the modular ratio.				—
6. Compression in bars other than twin twisted or other bent bars, in a beam or slab where the compressive resistance of the concrete is not taken into account.	18 000	Half guaranteed yield point, but not more than 20 000.			—

steel since it depends on the modulus of elasticity of the steel, which does not vary considerably. The use of deformed bars may restrict the opening of wide cracks if the member is seriously overloaded, but usually has little effect on the cracking at normal working loads.

In the case of solid slabs when the percentage of steel is low, the concrete in the tensile zone will contribute appreciably to the tensile resistance, and in this case the limiting stress of 27,000 lb. per square inch may safely be adopted (provided the yield stress is at least 54,000 lb. per square inch) without fear of serious cracking. In the case of heavily-reinforced beams, however, it may be desirable to limit the stress further when the beams are used in positions specially liable to give rise to corrosion. In such circumstances the use of mild steel at a stress of 18,000 lb. or 20,000 lb. per square inch will often be preferable to the use of higher tensile steels at correspondingly greater stresses.

(*c*) INCREASES OF PERMISSIBLE STRESS DUE SOLELY TO WIND FORCES

The maximum permissible stresses in reinforced concrete and in its reinforcement may exceed those given in (*a*) and (*b*) above respectively by not more than 33⅓ per cent provided :

(i) that such excess is solely due to stresses induced by wind loading, and
(ii) that in no case does the stress in the reinforcement exceed 27 000 lb./sq. in.

This concession, with regard to stresses due solely to wind, is common to most types of structural systems. The temporary overstress is not serious if limited so as to preclude (i) the possibility of failure due to fatigue and (ii) the opening of wide cracks which will not close sufficiently when the wind ceases. The limitation of the increase in stress to 33⅓ per cent. of the normal values is sufficient to deal with the first point ; the overall upper limit to the steel stress given in Clause 303 (*c*) (ii) ensures the necessary safeguard against excessive cracking.

304. Cover. Reinforcement should have concrete cover and the thickness of such cover (exclusive of plaster or other decorative finish) should be:—

(*a*) for each end of a reinforcing bar, not less than 1 in., nor less than twice the diameter of such rod or bar.

(*b*) for a longitudinal reinforcing bar in a column, not less than 1½ in., nor less than the diameter of such rod or bar. In the case of columns of minimum dimension of 7½ in. or under, whose bars do not exceed ½ in. diameter, 1 in. cover may be used.

(*c*) for a longitudinal reinforcing bar in a beam, not less than 1 in., nor less than the diameter of such rod or bar.

(*d*) for tensile, compressive, shear or other reinforcement in a slab, not less than ½ in., nor less than the diameter of such reinforcement.

(*e*) for any other reinforcement, not less than ½ in., nor less than the diameter of such reinforcement.

For all external work, for work against earth faces, and also for internal work where there are particularly corrosive conditions, the cover of the concrete should be increased by ½ in. beyond the figures given in (*a*) to (*e*) above.

In the case of bars which are not round, and twin bars, the diameter should be taken as the diameter of a circle giving an equivalent area.

The requirements for cover given in this clause are more onerous than in

The bond stress requirements relate to (*a*) the ***average*** bond stress along the whole length of the bar to the section considered, and (*b*) the ***local*** bond stress depending on the shear at the section. The local stress may be about 1¾ times the value permitted for the average bond stress (see *Tables* 4 and 5).

The lengths of bars to satisfy the requirements in Clause 307 (*a*) are given in *Tables* 6A and 7A. In *Table* 6A the length of bar, given in terms of the number of bar diameters, required beyond a section stressed in tension is given for the various concrete mixes. In *Table* 7A the lengths are converted into inches for the various sizes of bars normally used.

It should be noted that the lengths specified are to the end of the bar and include the anchorage length of twelve bar diameters required beyond the section at which the tension in the steel disappears. Where a standard hook is used the length to the end of the bar may be assumed to be four bar diameters less than the values given in *Table* 6A. Although the lengths given in *Table* 6A satisfy the Code requirements, they should be considered as absolute minima. Wherever practicable, these values should be increased by at least ten bar diameters.

Table 6A.—End Lengths in Bar Diameters.

	Nominal concrete mix	Average bond stress	Tensile Stress in bar in lb. per square					
			14,000	16,000	18,000	20,000	25,000	27,000
	Portland-cement Concretes							
Normal Grade	1 : 1 : 2	150	24	27	30	34	42	45
	1 : 1½ : 3	135	26	30	34	37	47	50
	1 : 2 : 4	120	30	34	38	42	53	57
Lower Grade	1 : 1 : 2	110	31	36	40	45	56	60
	1 : 1½ : 3	100	35	40	45	50	62	67
	1 : 2 : 4	90	39	45	50	56	70	75
	High-Alumina Cement Concrete							
	1 : 2 : 4	150	24	27	30	34	42	45

The local bond stress for members of varying depth may be calculated from the equation (see *Fig.* 5A, page 34),

$$s_b = \frac{S \pm \frac{M}{d} \tan \alpha}{ao},$$

the negative sign applying to sections in which the bending moment (M) increases numerically in the same direction as the effective depth (d) increases, and the positive sign to sections in which the bending moment decreases numerically in this direction.

In the case of simply-supported beams, the requirements for bond often control the proportions of reinforcement which must be carried through to the support. This can be investigated as follows.

TABLE 7A.—END ANCHORAGE.

END LENGTHS OF BAR (IN INCHES)

END LENGTH IN BAR DIAMETERS	DIAMETER OF BAR (INCHES)										
	½	⅝	¾	⅞	1	1⅛	1¼	1⅜	1½	1¾	2
12 *d* (minimum)	6	8	9	11	12	14	15	17	18	21	24
14 *d*	7	9	11	13	14	16	18	20	21	25	28
16 *d*	8	10	12	14	16	18	20	22	24	28	32
18 *d*	9	12	14	16	18	21	23	25	27	32	36
20 *d*	10	13	15	18	20	23	25	28	30	35	40
22 *d*	11	14	17	20	22	25	28	31	33	39	44
24 *d*	12	15	18	21	24	27	30	33	36	42	48
26 *d*	13	17	20	23	26	30	33	36	39	46	52
28 *d*	14	18	21	25	28	32	35	39	42	49	56
30 *d*	15	19	23	27	30	34	38	42	45	53	60
32 *d*	16	20	24	28	32	36	40	44	48	56	64
34 *d*	17	22	26	30	34	39	43	47	51	60	68
36 *d*	18	23	27	32	36	41	45	50	54	63	72
38 *d*	19	24	29	34	38	43	48	53	57	67	76
40 *d*	20	25	30	35	40	45	50	55	60	70	80
42 *d*	21	27	32	37	42	48	53	58	63	74	84
44 *d*	22	28	33	39	44	50	55	61	66	77	88
46 *d*	23	29	35	41	46	52	58	64	69	81	92
48 *d*	24	30	36	42	48	54	60	66	72	84	96
50 *d*	25	32	38	44	50	57	63	69	75	88	100
52 *d*	26	33	39	46	52	59	65	72	78	91	104
54 *d*	27	34	41	48	54	61	68	75	81	95	108
56 *d*	28	35	42	49	56	63	70	77	84	98	112
58 *d*	29	37	44	51	58	66	73	80	87	102	116
60 *d*	30	38	45	53	60	68	75	83	90	105	120
62 *d*	31	39	47	55	62	70	78	86	93	109	124
64 *d*	32	40	48	56	64	72	80	88	96	112	128
66 *d*	33	42	50	58	66	75	83	91	99	116	132
68 *d*	34	43	51	60	68	77	85	94	102	119	136
70 *d*	35	44	53	62	70	79	88	97	105	123	140
72 *d*	36	45	54	63	72	81	90	99	108	126	144
74 *d*	37	47	56	65	74	84	93	102	111	130	148
76 *d*	38	48	57	67	76	86	95	105	114	133	152
78 *d*	39	49	59	69	78	88	98	108	117	137	156
80 *d*	40	50	60	70	80	90	100	110	120	140	160
82 *d*	41	52	62	72	82	93	103	113	123	144	164

Let the maximum bending moment in the span be $\frac{Wl}{K_1}$ and the shearing force at the support be $\frac{W}{K_2}$. Then $\frac{Wl}{K_1} = tA_s a = tN_1\frac{\pi d^2}{4}a$, or $N_1 = \frac{4Wl}{K_1 ta.\pi d^2}$, where N_1 is the number of bars of diameter d required for bending. Also, $\frac{W}{K_2} = s_b ao = s_b a N_2.\pi d$, or $N_2 = \frac{W}{K_2 s_b a.\pi d}$, where N_2 is the number of bars of diameter d required for bond.

Therefore
$$\frac{N_2}{N_1} = \frac{K_1}{K_2}.\frac{td}{4s_b l}.$$

Taking the case of uniformly-distributed load where $K_1 = 8$ and $K_2 = 2$, and where the stress in the reinforcement is 18,000 lb. per square inch and the permissible bond stress is 180 lb. per square inch (1 : 2 : 4 Portland-cement concrete), $\frac{N_2}{N_1} = 100\frac{d}{l}$. If the bar diameter ($d$) is 1 in. and the span (l) is 15 ft. $\left(\frac{d}{l} = \frac{1}{180}\right)$, it will be seen that $\frac{N_2}{N_1} = \frac{5}{9}$, and that just over one-half of the bars required at mid-span must be carried to the support. It will be noted that for given conditions of bending moment, shearing force, and permissible stresses, the proportion of bars to be carried to the support depends directly on the ratio $\frac{d}{l}$. It should also be noted that all the bars carried through to the supports have to resist tensile forces and must therefore be anchored at the ends, with a hook or a length of bar equal to twelve bar diameters, beyond the centre line of the support.

Where local bond stresses induced by bending are of high intensity and extend over a considerable length of bar, it may be necessary to increase the anchorage in order to satisfy the requirements of Clause 307 (*a*). In cases where shear is estimated to be taken by inclined compression at the support, the anchorage of the tensile bars should be adequate to enable such compression to be developed.

In the case of continuous beams it is advisable to carry some of the tensile reinforcement in the bottom of the beam at mid-span right across the span to

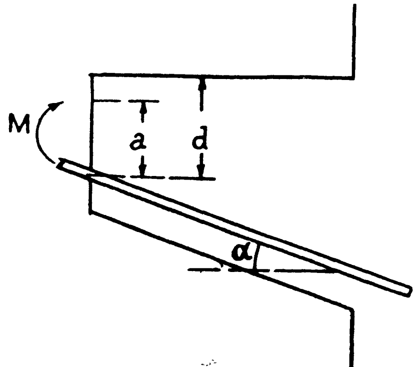


FIG. 5A.

the support. As a general rule this should not be less than one-third of the number of bars at mid-span, but in most cases the necessity of providing compressive reinforcement near the supports leads to the adoption of an arrangement both satisfactory and practical.

A hook will not be effective in places where inadequate cover is likely to lead to splitting of the concrete. At the sides of the hook and at its end the cover should be at least twice the diameter of the bar, while above the hook it should be at least three times the diameter of the bar. These values may be reduced for concretes with a Works-cube strength over 3000 lb. per square inch. Where this cover is provided no further precautions need be taken against splitting.

Where wrapping reinforcement is used to prevent splitting it should be arranged so that the concrete can be thoroughly consolidated inside the hook and around the reinforcement, otherwise it will serve no useful purpose and may even reduce the effectiveness of the hook.

The reference to " other anchorages " in Clause 307 (*c*) presumably implies that special anchorages are permissible if they have an anchorage value equivalent to the resistance produced by the permissible bond stress acting over a length of bar equal to 12 times the diameter of the bar. The local stress permitted in the concrete at special anchorages is not stated, but may presumably be assumed to be three times the value permitted for concrete in direct compression, as in the case of bends [Clause 307 (*e*)]. The area on which this stress acts should not exceed one-twentieth of the cross-sectional area of the concrete, and the cover should be adequate in all directions. If these conditions cannot be complied with, effective wrapping reinforcement should be provided.

> (*e*) *Bearing stresses in bends.* In bends in reinforcing bars, the local stress on the concrete may be increased to three times the value permitted in clause 303 for the concrete in direct compression.

This requirement affects bars which may be bent up in beams, bars in retaining walls, and in other parts of a structure where bending is necessary or desirable. The local stress in the concrete is dependent on the stress (t) in the bar at the point of bend (which may be less than the maximum) and on the radius of curvature expressed in bar diameters $\left(\text{that is } \frac{R}{d}\right)$. The local stress is $t\frac{\pi}{4}.\frac{d}{R}$. The minimum radius of the bend is thus given by $\frac{R}{d} = t\frac{\pi}{4}.\frac{1}{3c}$, where c is the permissible stress in the concrete in direct compression, that is $\frac{R}{d} = 0{\cdot}26\frac{t}{c}$.

Table 8A gives the radii necessary for bars at various stresses with concretes of different strengths.

TABLE 8A.—BENDS IN BARS.

Radii of bends in bar diameters $\left(\frac{R}{d} = 0\cdot26\frac{t}{c}\right)$.

	Nominal concrete mix	Direct stress lb. per sq. in.	Stress t (in lb. per square inch)							
			10,000	12,000	14,000	16,000	18,000	20,000	25,000	27,000
			R	*R*	*R*	*R*	*R*	*R*	*R*	*R*
			Portland cement concretes							
Normal Grade	1 : 1 : 2	1140	3*d*	3*d*	4*d*	4*d*	5*d*	5*d*	6*d*	7*d*
	1 : 1½ : 3	950	3*d*	4*d*	4*d*	5*d*	5*d*	6*d*	7*d*	8*d*
	1 : 2 : 4	760	4*d*	5*d*	5*d*	6*d*	7*d*	7*d*	9*d*	10*d*
Lower Grade	1 : 1 : 2	850	3*d*	4*d*	5*d*	5*d*	6*d*	7*d*	8*d*	9*d*
	1 : 1½ : 3	710	4*d*	5*d*	6*d*	6*d*	7*d*	8*d*	10*d*	10*d*
	1 : 2 : 4	570	5*d*	6*d*	7*d*	8*d*	9*d*	10*d*	12*d*	13*d*
			High-alumina cement concrete							
	1 : 2 : 4	1140	3*d*	3*d*	4*d*	4*d*	5*d*	5*d*	6*d*	7*d*

(*f*) *Secondary reinforcements.* Notwithstanding any of the provisions of this code, in the case of secondary reinforcements such as stirrups and binding, complete bond length and anchorage may be deemed to have been provided when a bend in the bar through an angle of at least 90 degrees passes round a bar of at least its own diameter and the stirrup or binding is continued beyond the end of the curve for a length of at least eight diameters.

(*g*) *Bars in compression.* The length of bar beyond any section should not be less than n times the diameter of the bar where

$$n = \frac{\text{the compressive stress in the bar}}{\text{five times the permissible bond stress } B_1}$$

(*h*) *Deformed bars.* In the case of deformed bars, the bond stresses in Tables 4, 5 and 6 may be increased by 10 per cent.

For the purpose of this clause, a deformed bar is one for which it has been proved that the bond strength exceeds that of a plain round bar by 10 per cent or more.

BEAMS AND SLABS

308. General. (*a*) *Effective span.* The effective span, *l*, of a beam or slab should be taken as the lesser of the two following:—

(i) the distance between the centres of supports ; or

(ii) the clear distance between supports plus the effective depth of the beam or slab, the effective depth being the distance between the centre of tension and the edge of the compression section.

(*b*) *Slender beams.* Where the length L of a beam between adequate lateral restraints exceeds 20 times the breadth B of its compression flange, the maximum compressive stress in the concrete should not exceed the product of the permissible compressive stress due to bending given in clause 303, and the appropriate coefficient given in Table 8 below.

TABLE 8.—STRESS REDUCTION FACTORS FOR SLENDER BEAMS.

Slenderness ratio L/B	20	30	40	50	60
Coefficients*	1·00	0·75	0·50	0·25	0

* Intermediate values may be obtained by linear interpolation.

This clause will be operative principally for beams of rectangular section. The length of the beam is the part unsupported laterally, which may or may not be the entire length between the main vertical supports.

(*c*) *Minimum reinforcement in slabs.* In solid reinforced concrete slabs the reinforcement should be not less in either direction than 0·1 per cent of the gross cross-sectional area of the concrete.

Floor and roof slabs are usually supported by parallel beams with others at right angles, so forming a series of adjacent square or rectangular panels. Frequently the latter are of such a length that the panel effect is negligible, and practically the whole of the load is carried by the longitudinal supports. Uniformly-loaded panels having a length-to-width ratio of three or more are in this category, and in such cases no calculations need be made for bending in the longitudinal direction.

There are, however, many ways in which a long panel can in practice be loaded so as to produce appreciable bending in the longitudinal direction, and consequently reinforcement must be provided to resist the tensile stresses thereby induced. A load of an intensity much greater than that for which a slab has been nominally designed but occupying a relatively small area near the centre of the panel is a case in point. Where such a load occurs upon a slab, longitudinal bending will be produced no matter how it is supported; in fact it is unlikely in practice that a slab would be supported and loaded so as to produce bending in one direction only. The amount of longitudinal bending produced will be relatively small where slabs span between parallel beams or in long panels, and the provision of 0·1 per cent. of distribution reinforcement will often be sufficient to meet the requirements for concentrated loads, local shrinkage, and other secondary effects neglected in the calculations. This amount of distribution steel is, however, an absolute minimum which may not be sufficient for heavily-loaded floors.

The Code of Practice CP 114.101 for Concrete Floors and Roofs of Solid Slab Construction, published after the main Code CP 114, contains the provision that the transverse reinforcement should have a cross-sectional area of not less than 10 per cent. of that of the main reinforcement. This requirement should be satisfied in addition to that of Clause 308 (*c*). The maximum spacings of main and distribution bars are also limited by Clause 305.

It should be mentioned that, in long panels subjected to a uniformly-distributed load, longitudinal bending occurs locally at each end of the panel. The positive bending moments produced by this local bending are greater than along the central portion of the panel, and for a particular short span these bending moments remain approximately constant for all ratios of length to width in excess of two. Distribution reinforcement equal to 10 per cent. of the sectional area of the main bars is sufficient to provide for this local bending in long panels. The same amount of negative reinforcement should be provided over the short end supports.

Tables 9A and 10A give the spacing of distribution bars to give an area per unit width of slab equal to one-tenth of that for the main bars. *Table* 9A gives the pitch of distribution bars in relation to that adopted for the main bars for the sizes most commonly in use. In *Table* 10A these relative spacings are converted into actual values in inches for various spacings of the main bars.

TABLE 9A.—DISTRIBUTION REINFORCEMENT IN SLABS.

This table gives the pitch of distribution bars in terms of the spacing of the main bars to satisfy 10 per cent. requirement of Code CP 114.101. Table 10A translates this spacing into inches.

Dia. of main bars (in.)	SPACING FACTORS								
	Diameter of transverse bars (in.)								
	3/16	1/4	5/16	3/8	1/2	5/8	3/4	7/8	1
3/16	10·0								
1/4	5·62	10·0							
5/16	3·60	6·40	10·0						
3/8	2·50	4·44	6·94	10·0					
1/2	1·40	2·50	3·90	5·62	10·0				
5/8	0·90	1·60	2·50	3·60	·40	10·0			
3/4		1·11	1·73	2·50	4·44	6·94	10·0		
7/8			1·27	1·83	3·26	5·10	7·34	10·0	
1			0·97	1·40	2·50	3·90	5·62	7·65	10·0

(*d*) *Compression reinforcement in beams.* The compression reinforcement should be effectively anchored in two directions at right angles over the distance where it is required to act in compression, at points not further apart, centre to centre, than twelve times the diameter of the anchored bar. The subsidiary reinforcement used for this purpose should pass round, or be hooked over, both the compressive and tensile reinforcement.

The percentage of steel in compression should preferably not exceed 4 per cent but, if it does, only 4 per cent should be allowed for in the calculation of the resistance moment of the beam.

Top compressive reinforcement may be required in the middle of the span, and for cantilevers and continuous beams compressive reinforcement is usually

TABLE 10A.

Spacing of main bars (in.)	DISTRIBUTION BARS: SPACING IN INCHES																			
	Spacing Factors from Table 9A																			
	0·90	0·97	1·11	1·27	1·40	1·60	1·73	1·83	2·50	3·26	3·60	3·90	4·44	5·10	5·62	6·40	6·94	7·34	7·65	10·0
2	1½	1½	2	2½	2½	3	3	3½	5	6½	7	7½	8½	10	11	12	13	14	15	20
2½	2	2	2½	3	3½	4	4	4½	6	8	9	9½	11	12	14	16	17	18	19	25
3	2½	2½	3	3½	4	4½	5	5	7½	9½	10	11	13	15	16	19	20	22	22	30
3½	3	3	3½	4	4½	5½	6	6	8½	11	12	13	15	17	19	22	24	25	26	35
4	3½	3½	4	5	5½	6	6½	7	10	13	14	15	17	20	22	25	27	29	30	40
4½	4	4	5	5½	6	7	7½	8	11	14	16	17	20	22	25	28	31	33	34	45
5	4½	4½	5½	6	7	8	8½	9	12	16	18	19	22	25	28	32	34	36	38	50
5½	4½	5	6	6½	7½	8½	9½	10	13	17	19	21	24	28	30	35	38	40	42	55
6	5	5½	6½	7½	8	9½	10	11	15	19	21	23	26	30	33	38	41	44	45	60
7	6	6½	7½	8½	9½	11	12	12	17	22	25	27	31	35	39	44	48	51		
8	7	7½	8½	10	11	12	13	14	20	26	28	31	35	40	44	51				
9	8	8½	10	11	12	14	15	16	22	29	32	35	40	45	50					
10	9	9½	11	12	14	16	17	18	25	32	36	39	44	51						
11	9½	10	12	13	15	17	19	20	27	35	39	42	48	56						
12	10½	11	13	15	16	19	20	22	30	39	43	46	53							

necessary in the lower portion of the beam adjacent to the supports. Compressive forces near the supports usually require the provision of longitudinal reinforcement equal in area to the tensile bars provided. With rectangular beams this may apply equally at mid-span, but for T-beams the greater area of concrete available generally enables the compressive reinforcement to be reduced and often omitted.

The requirement that the reinforcement shall be anchored laterally and vertically should be interpreted with discretion. For example, a single stirrup passing around the whole group of bars may in suitable circumstances reasonably be held to anchor the reinforcement effectively.

With T-beams top reinforcement in the form of " clamps " of bent-up bars will be present in the slab and this, being at right-angles to and above the longitudinal compressive bars, provides anchorage which may be sufficient apart from any additional restraint provided by the beam stirrups or other shear reinforcement.

(*e*) *T-beams.* In T-beams the breadth of the flange assumed as taking compression should not exceed the least of the following :—

(i) one-third of the effective span of the T-beams ;
(ii) the distance between the centres of the ribs of the T-beams ;
(iii) the breadth of the rib plus twelve times the thickness of the slab.

(*f*) *L-beams.* In L-beams the breadth of the flange assumed as taking compression should not exceed the least of the following :—

(i) one-sixth of the effective span of the L-beams ;
(ii) the breadth of the rib plus one-half of the clear distance between ribs;
(iii) the breadth of the rib plus four times the thickness of the slab.

When a part of a slab is considered as the flange of a T-beam or L-beam, the reinforcement in the slab transverse to the beam should cross the full breadth of the flange. Where the slab is assumed to be spanning independently in the same direction as the beam, such transverse reinforcement should be near the top surface of the slab.

The quantity of such reinforcement should be related to the shear stress in the slab produced by its acting as the compression member of the T-beam or L-beam.

The requirement with regard to the amount of reinforcement to be provided in the slab transversely to the beam is not clear. It may normally be assumed that, where the slab spans on to a beam, the reinforcement provided in the slab to resist bending will be sufficient to develop the proper interaction between the rib and the flange. Where the slab spans in a direction parallel to the beam, " clamps " or other top reinforcement should be provided and a reasonable area would be at least 0·3 per cent. of the gross cross-sectional area of the slab.

(*g*) *Expansion joints.* It is recommended that the question of the provision of expansion joints should be left to the discretion of the reinforced concrete designer, owing to the large number of factors which are involved.

309. Bending moments.

(*a*) GENERAL.

Bending moments in beams and slabs should be calculated for the effective span and all loading thereon.

The bending moments to be provided for at a cross-section of a continuous beam or slab should be the maximum positive and negative moments at such cross-section, allowing, if so desired, for the reduced moments due to the width of the supports in both cases, for the following arrangements of superimposed loading :—

(i) alternate spans loaded and all other spans unloaded ;
(ii) any two adjacent spans loaded and all other spans unloaded.

Nevertheless, the maximum positive and negative moments so obtained may be modified by an amount not exceeding 15 per cent provided that any decrease in assumed moment at a support is accompanied by an increase of the same numerical value of the maximum moment in both adjacent spans, and that any decrease in assumed maximum span moment is accompanied by an increase of the same numerical value to the moment at the supports at both ends of the span.

In the case of an end span, where the end support is virtually pin-jointed, the maximum positive moment near mid-span may be increased or decreased by an amount not exceeding 7½ per cent provided that the moment at the first interior support is decreased or increased respectively by twice the numerical value of the adjustment of the maximum positive moments near mid-span.

The increase to the positive moments required under the two foregoing paragraphs should be moments which occur in the conditions of loading which apply when the maximum negative moment occurs, and vice versa.

Moments at other sections should be adjusted accordingly.

The computation of bending moments in flat slabs is dealt with in clauses 316 and 320.

The conditions specified for the loading of beams and slabs give the maximum theoretical positive bending moments in the span, but less than the maximum negative theoretical bending moments at the supports. Taking into account the further concession that the calculated " support " moments may be reduced (subject to the moments in adjacent spans being similarly increased), the amount of reinforcement required to resist negative bending moments at the supports is very considerably less than that found by theoretical calculations based upon the usual assumptions. This reduction of support moments has the great merit of reducing the congestion of reinforcement at the supports where the satisfactory consolidation of the concrete is so often a practical difficulty, and is justified by several factors ignored in these assumptions, particularly the redistribution of bending moments that occurs before failure as a result of inelastic deformation of the concrete or steel. Such redistribution of bending moments occurs whenever incipient failure is reached, whether at a support or in the span, and the Code therefore allows a reduction in the bending moment on the span also to be made if the moments at the adjacent supports are increased by the same numerical value.

Clause 309 (*a*) includes the important provision that the increases in bending moments to accompany an assumed reduction are to be made for *corresponding* bending moments for each particular loading considered. That is, when considering the maximum negative bending moment to be used in the design of a certain support, the adjacent spans on each side of the support are considered to be loaded and all other spans unloaded (except for dead load). The calculated bending moment at the support may then be reduced by 15 per cent.

provided that the bending moments in each adjacent span *as calculated for this same loading* are increased by the same amount.

Similarly, when considering the maximum positive bending moment for the design of a particular span, that span and alternate spans are considered to be loaded. The calculated positive bending moment on the span may then be reduced by 15 per cent. provided that the negative bending moments at adjacent supports are increased by the same numerical amount.

Since the bending moments on the spans are comparatively small when the conditions of loading are such that they give a maximum bending moment at the support, and, on the other hand, the bending moments at the supports are comparatively small when the conditions of loading are such that they produce a maximum bending moment on the span, it usually follows that allowance for redistribution of bending moments leads to design moments at the supports which are 15 per cent. less than the maximum values calculated, together with bending moments on the span that are also less than the maximum values. This is shown in *Fig.* 6A, in which curve (1) represents the bending moments for spans AB and BC loaded and other spans not loaded; curve (2) represents the bending moments for span AB and alternate spans loaded; curve (3) represents the bending moments for span BC and alternate spans loaded. The heavy line indicates the bending-moment envelope to be assumed for design. It is suggested that the negative bending moment should not be altered except close to the supports, and that the reduction in positive bending moments below the maximum values should be made in proportion at all points. This will provide a satisfactory and simple adjustment easy to apply and providing an ample reserve of strength.

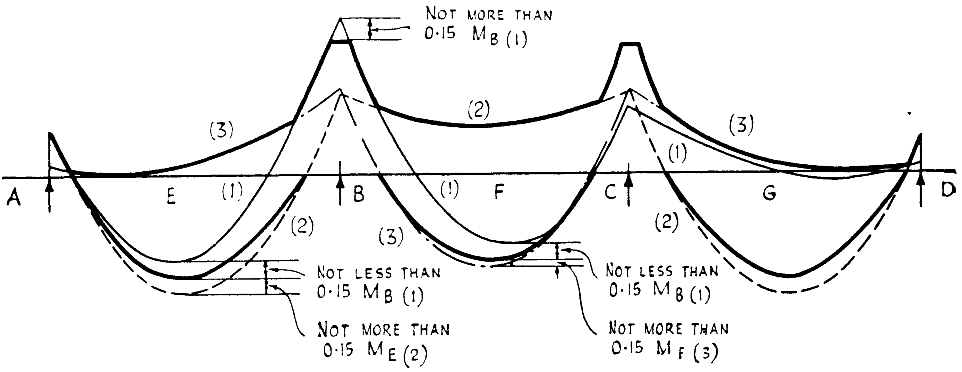

NOT MORE THAN 0·15 M_B (1)
(3)
(2)
(3)
(1)
A
E
(1)
B
(1)
F
C
G
D
(2)
(3)
(2)
NOT LESS THAN 0·15 M_B (1)
NOT MORE THAN 0·15 M_E (2)
NOT LESS THAN 0·15 M_B (1)
NOT MORE THAN 0·15 M_F (3)

FIG. 6A.—TYPICAL BENDING-MOMENT CURVES.

In the case of end spans, the reduction in the positive bending moment is limited to 7½ per cent. The bending moment at the first interior support may be reduced in one of two ways: a reduction of 15 per cent. is permissible if the bending moments on the adjacent spans (corresponding to the same loading) are increased by the same numerical value; alternatively, a reduction limited to 15 per cent. of the numerical value of the bending moment on the end span may be made with an increase of 7½ per cent. of this moment in the end span and an increase of 15 per cent. of this moment in the second span.

For small beams and slabs the lengths of the bars would not be affected by the percentage variations, the bottom bars being either all carried on to the support or some being stopped off in the customary manner.

For main beams containing a large amount of reinforcement it is usual to calculate the envelope of maximum bending moments obtained when the spans are loaded in the alternative ways specified. For these cases the adjustments will presumably be made to the diagram and reinforcement provided accordingly.

It should be noted that the percentage additions affect the slopes of the curves and therefore the intensities of shear. Since this adjustment is arbitrary, and in any case the effect on shear is small, no change need be made to the calculations for shear.

Although not specifically mentioned, the reduction of 15 per cent. of the bending moments at the support will apply to a beam with fixed ends, subject, of course, to the bending moment on the span being similarly increased. For a fixed beam uniformly loaded the resultant maximum positive and negative bending moments would be $+\frac{13Wl}{240}$ and $-\frac{17Wl}{240}$ respectively.

Bending moment coefficients for continuous beams with uniform moments of inertia for various numbers of equal spans are given in *Fig.* 7A for dead load and in *Fig.* 8A for the superimposed load specified in Clause 309 (*a*). The coefficients given in brackets are for the bending moments after the adjustments for redistribution of the bending moments have been made: these adjustments can be varied slightly whilst still conforming to the Code requirements, and the values given are those applicable when maximum reductions of the bending moments at the supports are desired.

It should be noted that the coefficients are based on the assumption that the beams are continuous over the supports and are capable of free rotation about them, with the ends of the beams freely supported. Where the ends of continuous beams frame into columns or other members, the bending moments obtained from these coefficients will, of course, be affected. The moments induced at the intermediate supports of a continuous beam, when a unit moment is applied at the end support, are given in *Fig.* 9A.

(*b*) BEAMS AND SLABS SPANNING IN ONE DIRECTION.

The bending moments in beams and slabs spanning in one direction may be calculated on one of the following assumptions :—

(i) beams may be designed as members of a continuous framework, with monolithic connection between the beams and columns, and the bending moments calculated taking into account the resistance of the columns to bending. Where beams are framed into external columns they should be designed to resist bending moments in combination with the columns in conformity with clause 312 (*b*), or

(ii) beams and slabs may be designed as continuous over supports and capable of free rotation about them. Nevertheless, where the supports to beams or slabs are monolithic with them and stiff in relation to them, it is preferable to design the beams or slabs with due regard to such stiffness.

For the purpose of calculating moments in beams or slabs in a monolithic structure, it will usually be sufficiently accurate to assume that members

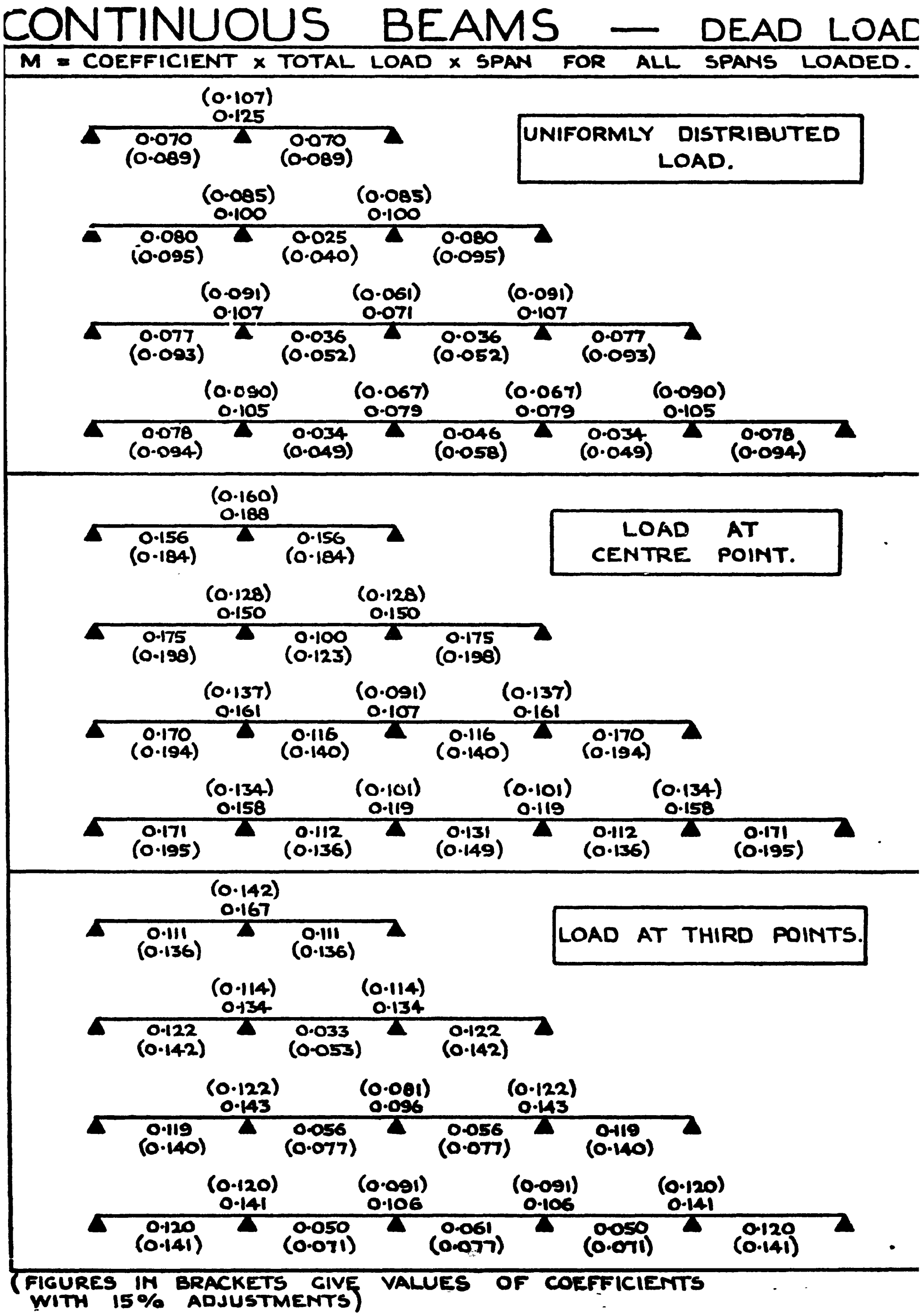
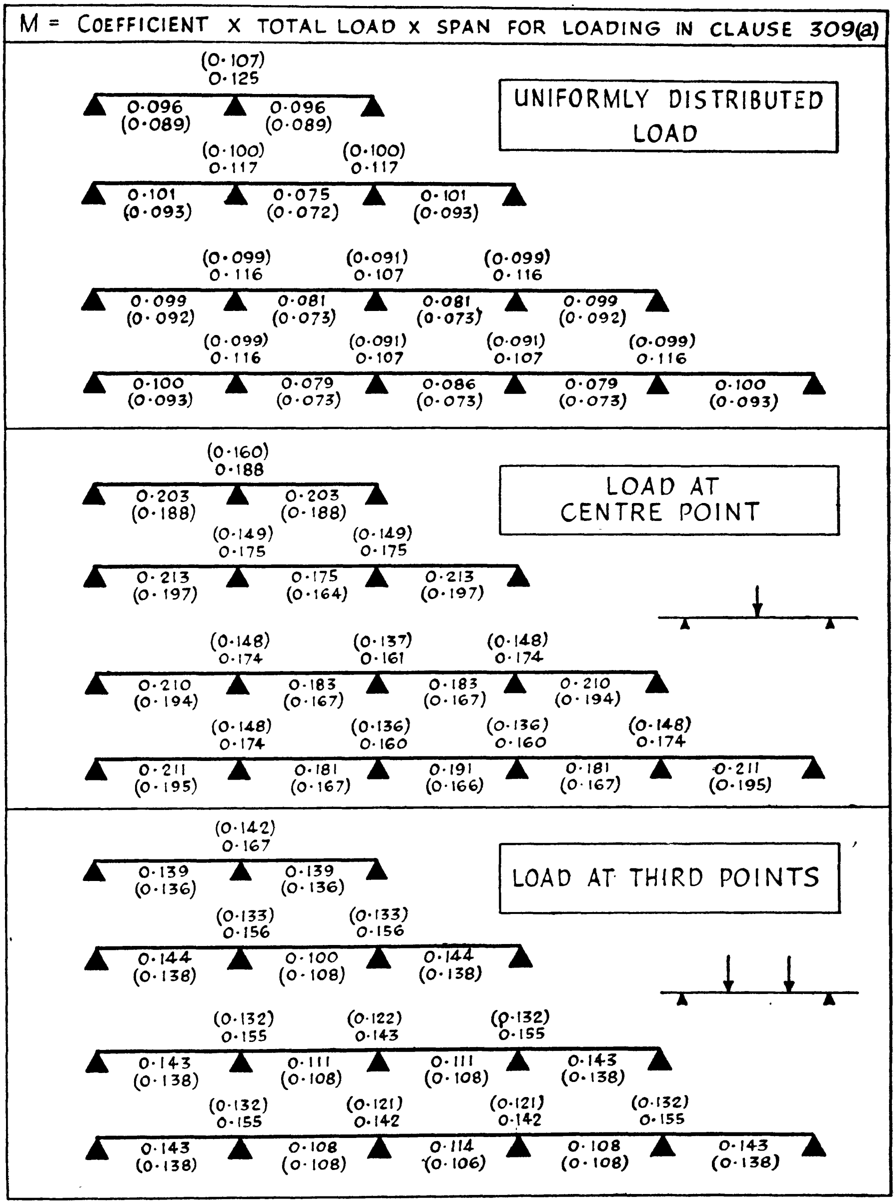

CONTINUOUS BEAMS — DEAD LOAD
M = COEFFICIENT x TOTAL LOAD x SPAN FOR ALL SPANS LOADED.
UNIFORMLY DISTRIBUTED LOAD.
(0·107)
0·125
0·070
(0·089)
0·070
(0·089)
(0·085) (0·085)
0·100 0·100
0·080 0·025 0·080
(0·095) (0·040) (0·095)
(0·091) (0·061) (0·091)
0·107 0·071 0·107
0·077 0·036 0·036 0·077
(0·093) (0·052) (0·052) (0·093)
(0·090) (0·067) (0·067) (0·090)
0·105 0·079 0·079 0·105
0·078 0·034 0·046 0·034 0·078
(0·094) (0·049) (0·058) (0·049) (0·094)
LOAD AT CENTRE POINT.
(0·160)
0·188
0·156 0·156
(0·184) (0·184)
(0·128) (0·128)
0·150 0·150
0·175 0·100 0·175
(0·198) (0·123) (0·198)
(0·137) (0·091) (0·137)
0·161 0·107 0·161
0·170 0·116 0·116 0·170
(0·194) (0·140) (0·140) (0·194)
(0·134) (0·101) (0·101) (0·134)
0·158 0·119 0·119 0·158
0·171 0·112 0·131 0·112 0·171
(0·195) (0·136) (0·149) (0·136) (0·195)
LOAD AT THIRD POINTS.
(0·142)
0·167
0·111 0·111
(0·136) (0·136)
(0·114) (0·114)
0·134 0·134
0·122 0·033 0·122
(0·142) (0·053) (0·142)
(0·122) (0·081) (0·122)
0·143 0·096 0·143
0·119 0·056 0·056 0·119
(0·140) (0·077) (0·077) (0·140)
(0·120) (0·091) (0·091) (0·120)
0·141 0·106 0·106 0·141
0·120 0·050 0·061 0·050 0·120
(0·141) (0·071) (0·077) (0·071) (0·141)
(FIGURES IN BRACKETS GIVE VALUES OF COEFFICIENTS WITH 15% ADJUSTMENTS)

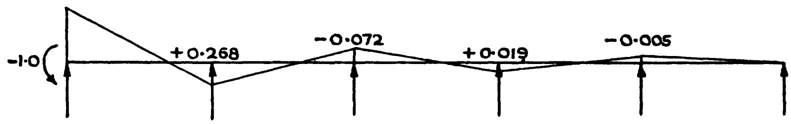
FIG 7A

CONTINUOUS BEAMS — SUPERIMPOSED LOAD



(FIGURES IN BRACKETS GIVE VALUES OF COEFFICIENTS WITH ADJUSTMENTS FOR MOMENT REDISTRIBUTION) — CLAUSE 309 (a).

FIG. 8A.



NOTE:- LITTLE CHANGE FOR OTHER NUMBERS OF SPANS.

FIG. 9A.—MOMENTS AT INTERMEDIATE SUPPORTS, DUE TO END MOMENT.

connected to the ends of such beams or slabs are fixed inposition and direction at the ends of such members remote from their connections with the beam or slab.

The alternative methods given for the calculation of the bending moments in beams and slabs give different results but both lead to a satisfactory margin of safety, or load factor. The first method, in which beams may be considered as members of a continuous framework monolithic with the columns, is fully dealt with in many text-books on reinforced concrete design to which the reader is referred for guidance. It should be noted, however, that the adjustment of bending moments previously mentioned may be made if desired.

The design of a continuous framework to allow for the various conditions of loading that may occur is considerably simplified by the concession in Clause 309 (*b*) that the only part of the framework which need be considered in the design of a particular beam consists of the beam itself and the columns and beams framing into the ends of the beam, with conditions of complete fixity at the boundaries of this system.

The second assumption [Clause 309 (*b*) (ii)]—that beams are continuous over supports about which they can freely rotate—is that used in obtaining the customary bending moment factors for equal spans and is also the basis of the graphical methods employed for obtaining bending-moment envelope diagrams. At end supports, where beams frame into columns, such beams should be assumed to be monolithic with the columns and a reduction of the positive bending moment on the adjacent span, as well as an adjustment to the other moments, may justifiably be made in these cases.

As it is unlikely that the amount of the negative bending moment occurring at the end of a beam will be known at the time of calculating the moments elsewhere along the beam, it may facilitate these calculations to adopt an arbitrary bending moment for such end supports. The moment may vary from about $-\frac{Wl}{120}$ to $-\frac{Wl}{12}$ according to whether the column into which the beam frames is relatively flexible or whether it is stiff enough to approximate to absolute fixity. In normal cases it should be sufficient to assume that this bending moment is $-\frac{Wl}{24}$, but where, by inspection, a more accurate factor can be obtained this should be adopted. The requisite adjustments to the bending moments in the spans and at the supports may be made by the use of the coefficients given in *Fig.* 9A. Any error between the moment as subsequently calculated and the value assumed is unlikely to affect the remaining bending moments seriously, though the section of the beam at the column must be capable of taking the bending moment as ascertained from the calculations for the columns.

Values of this moment for various ratios of column to beam stiffness are given in *Table* 30A. The adjustments to bending moments to allow for moment redistribution may be made also when this second assumption is used in designing beams.

In the case of secondary beams supported at their ends by main beams, the latter may normally be regarded as "free" supports. Nevertheless, a negative bending moment should be allowed for at end supports and reinforcement provided accordingly. A bending moment of $-\frac{Wl}{24}$ is suggested as being suitable for the purpose.

In *Table* 11A the "economic" percentages of tensile reinforcement (when both concrete and steel are stressed to the full permissible values) are given on the assumption that the compressed section is rectangular and is not reinforced. Corresponding values for the depth of the neutral axis and for the resistance-moment coefficient $R = \frac{M}{bd^2}$ are also given for the various grades of concrete and steel.

Table 11A.—Resistance-Moment Coefficients for Portland Cement Concretes, with Aggregates Complying with B.S. 882.

Grade of concrete	Nominal mix	Permissible compressive stress in concrete in bending lb. per sq. in.	($m = 15$) Tensile stress in Reinforcement								
			18,000 lb. per sq. in.			20,000 lb. per sq. in.			27,000 lb. per sq. in.		
			"Economic" percentage	$\frac{n}{d}$	R	"Economic" percentage	$\frac{n}{d}$	R	"Economic" percentage	$\frac{n}{d}$	R
Normal	1 : 1 : 2	1500	2·31	0·556	340	1·99	0·529	327	1·26	0·455	289
	1 : 1½ : 3	1250	1·77	0·510	265	1·51	0·484	254	0·95	0·410	221
	1 : 2 : 4	1000	1·26	0·455	193	1·07	0·429	184	0·66	0·357	157
Lower	1 : 1 : 2	1125·0	1·51	0·484	228	1·29	0·458	218	0·80	0·385	189
	1 : 1½ : 3	937·5	1·14	0·439	176	0·97	0·413	167	0·59	0·342	142
	1 : 2 : 4	750·0	0·80	0·385	126	0·68	0·360	119	0·41	0·294	99

$d =$ effective depth; $n =$ depth to neutral axis; $R = \frac{\text{Resistance moment}}{bd^2}$; $b =$ breadth.

(*c*) SLABS SPANNING IN TWO DIRECTIONS AT RIGHT ANGLES WITH UNIFORMLY DISTRIBUTED LOADS.

(i) *General.* In order to estimate the bending moments in a slab spanning in two directions at right angles, the slab may be assumed to act as a perfectly elastic thin plate, Poisson's ratio being assumed equal to zero.

The use of the elastic theory for estimating the bending moments in a slab spanning in two directions is of chief value for slabs with restraint on four sides [see comment on Clause 309 (*c*) (iii)].

(ii) *Slabs simply supported on four sides.* Where, in the case of a simply supported slab, adequate provision is not made to resist torsion at the corners of the slab and to prevent the corners from lifting, the bending moments at mid-span should be assumed to have the values given by the following equations :

$$M_x = Z_x \frac{wl_x^2}{8} \quad \dots\dots\dots \quad (1)$$

$$M_y = Z_y \frac{wl_y^2}{8} \quad \dots\dots\dots \quad (2)$$

where M_x and M_y are the bending moments at mid-span on strips of unit width and spans l_x and l_y respectively ;

w is the total load per unit area ;
l_y is the length of the longer side (see Fig. 2) ;
l_x is the length of the shorter side (see Fig. 2) ;
Z_x and Z_y are coefficients shown in Table 9.

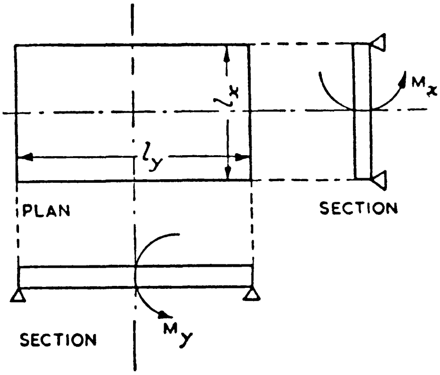


FIG. 2.—DIAGRAM SHOWING NOTATION USED FOR SLABS SUPPORTED ON FOUR SIDES.

TABLE 9.—BENDING MOMENT COEFFICIENTS FOR SLABS SPANNING IN TWO DIRECTIONS AT RIGHT ANGLES SIMPLY SUPPORTED ON FOUR SIDES.

$\frac{l_y}{l_x}$	1·0	1·1	1·2	1·3	1·4	1·5	1·75	2·0	2·5	3·0
Z_x	0·50	0·59	0·68	0·74	0·79	0·84	0·90	0·94	0·97	0·98
Z_y	0·50	0·41	0·32	0·26	0·21	0·16	0·10	0·06	0·03	0·02

The bending moment coefficients given in *Table* 9 are derived from the Grashof-Rankine formulæ which are based on the equality of deflection of the central orthogonal strips of the slab at their intersection. If the slab shown in *Fig.* 10A were subjected to a load w per unit area and were supported only by the longer supports, the bending moments at the centre of a strip of unit width would be $M'_x = \frac{wl_x^2}{8}$, and if it were supported only by the shorter sup-

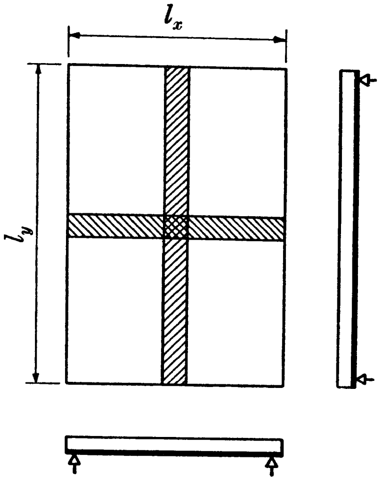


FIG. 10A.

ports the bending moment would be $M'_y = \frac{wl_y^2}{8}$. If now the slab be considered as simply supported on all four edges and the load be assumed to be divided between the two series of strips in such a way that the deflections of the two central strips are the same at the point where they cross, then $w = w_x + w_y$, and $\frac{5}{384}\cdot\frac{w_x l_x^4}{EI_x} = \frac{5}{384}\cdot\frac{w_y l_y^4}{EI_y}$, and, therefore, assuming that $I_x = I_y$, $w_x = w_y\frac{l_y^4}{l_x^4} = w\frac{l_y^4}{l_x^4 + l_y^4}$, and $w_y = w\frac{l_x^4}{l_x^4 + l_y^4}$, from which are obtained the Grashof-Rankine formulæ for the bending moments

$$M''_x = w\frac{l_y^4}{l_x^4 + l_y^4}\cdot\frac{l_x^2}{8} = Z_x\frac{wl_x^2}{8} \text{ or } Z_x M'_x,$$

$$\text{and } M''_y = w\frac{l_x^4}{l_x^4 + l_y^4}\cdot\frac{l_y^2}{8} = Z_y\frac{wl_y^2}{8} \text{ or } Z_y M'_y.$$

The bending moments obtained from this analysis are greater than would actually occur in a simply-supported slab. However, the exact calculation of the moment in such a slab in which the corners can lift is extremely complex. At the same time, neglect of torsion at the corners of the slab makes it desirable in design to use increased bending moments in the span. Such tests as have been carried out have indicated that the additional strength in the span obtained by the use of the coefficients of *Table* 9 influences the behaviour of the slab at the corners sufficiently to enable special corner reinforcement to be omitted.

(iii) *Slabs restrained on four sides.*

(1) Where the corners of a slab are prevented from lifting and adequate provision for torsion in accordance with (5) below is made, the bending moments may be assumed to have the values given in (3) below.

(2) Slabs are considered as being divided in each direction into middle strips and edge strips as shown in Fig. 3, the middle strip having a width of three-quarters the width of the slab and each edge strip having a width of one-eighth of the width of the slab, except that no edge strip should exceed $\frac{l_x}{2}$ in width.

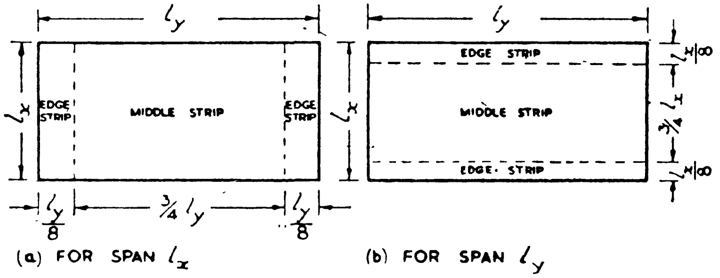


FIG. 3.—DIVISION OF SLAB INTO MIDDLE AND EDGE STRIPS.

(3) The maximum bending moments per unit width in the middle strip of a slab are given by the following equations :—

$$M_x = Z'_x w l_x^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

$$M_y = Z'_y w l_x^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

where M_x and M_y are the maximum bending moments on strips of unit width in the direction of spans l_x and l_y respectively ;

w is the total load per unit area ;
l_y is the length of the longer side ;
l_x is the length of the shorter side ;
Z'_x and Z'_y are coefficients given in Table 10.

(4) No reinforcement parallel to the adjacent edges of the slab need be inserted in the edge strips above that required to comply with clauses 305, 308 (*c*) and 309 (*c*) (iii) (5) below.

(5) Reinforcement should be provided at the corners of the slab. The effective area of such reinforcement per unit width should be equal to that required for the maximum positive moment in the middle strip, and this amount should be provided near both the top and bottom faces of the slab for a distance of one-fifth of the short span in both directions from the corner.

By 'the effective area of the reinforcement' is meant the normal area multiplied by the sine of the angle which the reinforcement makes with the critical sections. In the top of the slab the critical section is perpendicular to the diagonal. In the bottom of the slab it is parallel to the diagonal.

Any reinforcement which has been introduced to provide for the bending moments specified in 309 (*c*) (iii) (3) may be included as part of the reinforcement required to comply with this clause.

(6) Where a slab ends and there is monolithic connection between the slab and the supporting beam or wall, provision should be made for the negative moments that may occur in the slab at such support. The negative moment to be assumed in these cases depends on the degree of fixity afforded to the edge of the slab, but for general purposes it may be taken as two-thirds of the moment given in Table 10 for the mid-span of the slab.

Table 10.—Bending Moment Coefficients for Rectangular Panels supported on Four Sides with Provision for Torsion at Corners.

Type of panel and moments considered	Short span Z'_x								Long span Z'_y for all values of $\frac{l_y}{l_x}$
	Values of $\frac{l_y}{l_x}$								
	1·0	1·1	1·2	1·3	1·4	1·5	1·75	2·0 or more	
Case 1. Interior panels. Negative moment at continuous edge.	0·033	0·040	0·045	0·050	0·054	0·059	0·071	0·083	0·033
Positive moment at mid-span.	0·025	0·030	0·034	0·038	0·041	0·045	0·053	0·062	0·025
Case 2. One edge discontinuous. Negative moment at continuous edge.	0·041	0·047	0·053	0·057	0·061	0·065	0·075	0·085	0·041
Positive moment at mid-span.	0·031	0·035	0·040	0·043	0·046	0·049	0·056	0·064	0·031
Case 3. Two adjacent edges discontinuous. Negative moment at continuous edge.	0·049	0·056	0·062	0·066	0·070	0·073	0·082	0·090	0·049
Positive moment at mid-span.	0·037	0·042	0·047	0·050	0·053	0·055	0·062	0·068	0·037
Case 4. Two short edges discontinuous. Negative moment.	0·056	0·061	0·065	0·069	0·071	0·073	0·077	0·080	
Positive moment.	0·044	0·046	0·049	0·051	0·053	0·055	0·058	0·060	0·035
Case 5. Two long edges discontinuous. Negative moment.									0·056
Positive moment.	0·035	0·053	0·060	0·065	0·068	0·071	0·077	0·080	0·044
Case 6. Three edges discontinuous. Negative moment at continuous edge.	0·058	0·065	0·071	0·077	0·081	0·085	0·092	0·098	0·058
Positive moment at mid-span.	0·044	0·049	0·054	0·058	0·061	0·064	0·069	0·074	0·044
Case 7. Four edges discontinuous. Positive moment at mid-span.	0·050	0·057	0·062	0·067	0·071	0·075	0·081	0·083	0·050

The rules given in this clause are based on American regulations, which were derived from mathematical analyses by Professor Westergaard supplemented by test data. The effects of redistribution of bending moment have been allowed for and this has enabled the number of coefficients in *Table* 10 to be reduced to a minimum.

The simplification of the design procedure for a wide range of conditions at the edges is coupled with the requirement that reinforcement should be provided at the corners, and the amount of this reinforcement relative to that in the span is independent of the particular conditions at the edges. This requirement may be essential when the slab is discontinuous on all four sides, but is probably unnecessarily onerous for internal panels. For most practical cases of slab panels in buildings, even where one or more sides are discontinuous, the form of edge support is such as to provide a certain amount of fixity throughout its length. This partial fixity may be realised by the slab being monolithic with its supporting beam and suitably reinforced, by the slab entering a brick wall and being held down by the weight above, or by other means. Such practical fixity (provided that resistance to negative bending moment can be developed) has the effect of reducing the torsion at the corners, and proportionately the necessity for reinforcement at the corners. This effect cannot be allowed for if the simplified coefficients in *Table* 10 are adopted, but can be taken into account if the bending moments are calculated according to the theory of the elastic thin plate, as permitted by Clause 309 (*c*) (i).

When a slab is restrained along one or more of its edges, the mathematical investigation becomes involved and in many cases has not yet been worked out completely. By making certain simplifications, however, it is possible, with a reasonable degree of accuracy, to predict the bending moments in such panels.

DR. MARCUS'S METHOD.—The approximate method of slab design developed by Dr. H. Marcus in Germany,* and briefly outlined here, is similar in derivation to the Grashof-Rankine formulæ (see page 49) but introduces a simple correction to allow for restraint at corners and for the assistance given by torsion. It has been shown that the bending moments obtained in this simple manner vary by only 1 per cent. or 2 per cent. from those which have been obtained from more rigorous analyses based on the elastic plate theory.

The correction factors to the Grashof-Rankine formulæ for slabs simply supported on all four edges, given by Dr. Marcus, are $C_x = 1 - \frac{5}{6}.\frac{M''_x}{M'_y}$ and $C_y = 1 - \frac{5}{6}.\frac{M''_y}{M'_x}$, and the moments to be used in design at mid-span are therefore $M_x = C_x M''_x$ and $M_y = C_y M''_y$, which may be expressed as follows

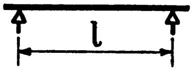
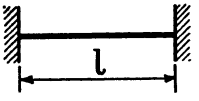
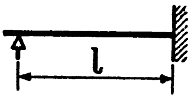
$$M_x = \left(1 - \frac{5}{6}.\frac{r^2}{1 + r^4}\right)M''_x$$

$$M_y = \left(1 - \frac{5}{6}.\frac{r^2}{1 + r^4}\right)M''_y$$

where $r = \frac{l_y}{l_x}$.

The procedure for slabs with any or all of the sides fixed is the same, the values of M''_x and M''_y being obtained from consideration of the strips according to the particular conditions of fixity obtaining as shown in *Table* 12A.

TABLE 12A.

			
DEFLECTION .	$\frac{5}{384}\frac{w_x l_x^4}{EI}$	$\frac{1}{384}\frac{w_x l_x^4}{EI}$	$\frac{2}{384}\frac{w_x l_x^4}{EI}$
MOMENT M''_x .	$\frac{1}{8} w_x l_x^2$	$\frac{1}{24} w_x l_x^2$	$\frac{9}{128} w_x l_x^2$

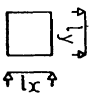
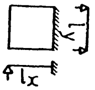
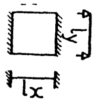
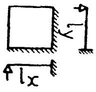
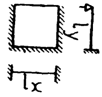
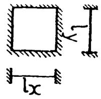
---

* "Die Theorie Elastischer Gewebe . . ." (Julius Springer, Berlin).

It should be noted that the deflections are taken at mid-span but that the values of M''_x and M''_y are maximum values for the span under consideration and are not necessarily at mid-span.

The six possible conditions of support, together with the appropriate values of the proportion of the load carried in each direction, and the expressions for C_x and C_y in terms of span ratios, are given in *Table* 13A.

TABLE 13A.

$r=\frac{l_y}{l}$	PROPORTION OF LOAD IN EACH DIRECTION		UNCORRECTED SPAN MOMENTS		VALUES OF COEFFICIENTS FOR SPAN MOMENTS	
CONDITIONS OF SUPPORT	In direction l_x $\frac{w_x}{w}$	In direction l_y $\frac{w_y}{w}=1-\frac{w_x}{w}$	M''_x	M''_y	C_x $(M_x=C_xM''_x)$	C_y $(M_y=C_yM'_y)$
I 	$\frac{r^4}{1+r^4}$	$\frac{1}{1+r^4}$	$\frac{1}{8}w_xl_x^2$	$\frac{1}{8}w_yl_y^2$	$1-\frac{5}{6}\frac{r^2}{1+r^4}$	$1-\frac{5}{6}\frac{r^2}{1+r^4}$
II 	$\frac{5r^4}{2+5r^4}$	$\frac{2}{2+5r^4}$	$\frac{9}{128}w_xl_x^2$	$\frac{1}{8}w_yl_y^2$	$1-\frac{75}{32}\frac{r^2}{2+5r^4}$	$1-\frac{5}{3}\frac{r^2}{2+5r^4}$
III 	$\frac{5r^4}{1+5r^4}$	$\frac{1}{1+5r^4}$	$\frac{1}{24}w_xl_x^2$	$\frac{1}{8}w_yl_y^2$	$1-\frac{25}{18}\frac{r^2}{1+5r^4}$	$1-\frac{5}{6}\frac{r^2}{1+5r^4}$
IV 	$\frac{r^4}{1+r^4}$	$\frac{1}{1+r^4}$	$\frac{9}{128}w_xl_x^2$	$\frac{9}{128}w_yl_y^2$	$1-\frac{15}{32}\frac{r^2}{1+r^4}$	$1-\frac{15}{32}\frac{r^2}{1+r^4}$
V 	$\frac{2r^4}{1+2r^4}$	$\frac{1}{1+2r^4}$	$\frac{1}{24}w_xl_x^2$	$\frac{9}{128}w_yl_y^2$	$1-\frac{5}{9}\frac{r^2}{1+2r^4}$	$1-\frac{15}{32}\frac{r^2}{1+2r^4}$
VI 	$\frac{r^4}{1+r^4}$	$\frac{1}{1+r^4}$	$\frac{1}{24}w_xl_x^2$	$\frac{1}{24}w_yl_y^2$	$1-\frac{5}{18}\frac{r^2}{1+r^4}$	$1-\frac{5}{18}\frac{r^2}{1+r^4}$

MOMENTS AT SUPPORTS.—According to the German regulations, in which Dr. Marcus's method is adopted, the bending moments at the supports are calculated as for slabs spanning in one direction only and the load used is the same as that employed for the calculations of the bending moments on the span. Thus, when two opposite edges are fixed, the average bending moment along these edges would be $-\frac{1}{12}w_xl_x^2$ or $-\frac{1}{12}w_yl_y^2$. When one edge is free and the opposite edge is fixed the bending moment would be $-\frac{1}{8}w_xl_x^2$ or $-\frac{1}{8}w_yl_y^2$. These moments are not the greatest that occur. In the case of a slab fixed on all sides the maximum value may be obtained approximately by dividing the average values by C_x (*Table* 13A, Case VI). The greatest value of this correction occurs for a square slab, and is about 15 per cent. Since, however, these bending moments occur over only a short length of the support, and the bending moments are less elsewhere along the support, it is quite safe to use the average value.

In the case of the shorter sides of long panels the bending moments at the support calculated in this way are very small. The same remark applies to the bending moments on the longitudinal spans of such panels, and it is considered desirable that increased values be taken in both these cases.

In *Tables* 14A to 24A the bending-moment factors are adjusted so that the moments $\left(\frac{wl_y^2}{K_y}\right)$ on the longitudinal span are in no case less than $\frac{wl_x^2}{50}$ and the moments $\left(-\frac{wl_y^2}{S_y}\right)$ at the supports in the same direction are in no case less than $-\frac{wl_x^2}{40}$. The adjusted factors are given in italics and are recommended for use.

Moments in Continuous Panels.—A method of estimating the bending moments produced on various systems of equal and continuous panels, based on the work of Dr. Marcus, is given by Professor B. Loser.* In this method calculations of the bending moments are made for two conditions of loading (*Fig.* 11A), namely

(*b*) the dead load of a panel together with one-half the live load are considered to act over all the panels, and

(*c*) one-half the live load is considered to act alternately upwards and downwards in consecutive panels.

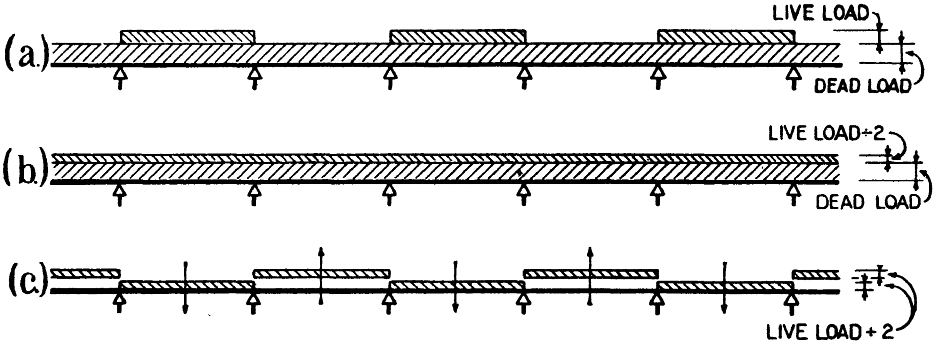


Fig. 11A.

The sums of the mid-span moments for the two conditions are then equal to the values that must be taken into account in design, that is, dead load on all panels and live load on alternate panels as in *Fig.* 11A (*a*). This method has the advantage that the bending moments for the two auxiliary conditions of loading can be obtained on the following assumptions: For the uniform load throughout (*b*), the panels may be considered as fixed at interior supports and freely supported at the boundary supports of the system; for the alternate upward and downward loading (*c*), each panel may be considered as simply supported.

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* "Bemessungsverfahren" (W. Ernst & Sohn, Berlin).

The method of computing the bending moments at the supports is as explained previously. Where there are only two panels in the direction under consideration, the bending-moment coefficient for the intermediate support is assumed to be one-eighth. Where there are three or more panels the bending moment at the first interior support is obtained from the proportion of load carried on the second and interior span, the coefficient being assumed to be one-tenth. For other interior supports the coefficient is taken as one-twelfth.

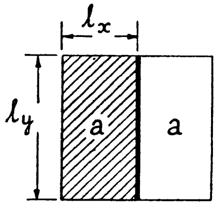
Tables 14A to 24A give bending-moment factors for various arrangements of rectangular panels continuous with panels of equal proportions as shown. The sketch at the head of each table shows the particular panel and supports to which the factors in the table relate.

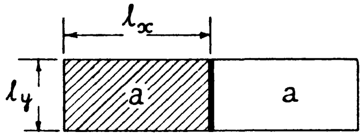
No reduction [Clause 309 (*a*)] should be made to the bending moments in two-way slabs, whether *Table* 10 or *Tables* 14A to 24A are used, since both methods are based upon approximate factors in which suitable allowances for redistribution of bending moments have been made.

Point Loads.—A method of treating concentrated loads on slab panels is given in a series of articles by W. L. Scott in " Concrete and Constructional Engineering " for March, April, and May, 1930.

TABLE 14A.—SLAB PANELS.

BENDING MOMENT FACTORS FOR DISTRIBUTED LOAD.





TWIN PANEL.

SHAPE OF PANEL $\frac{l_y}{l_x}$	END SPAN l_x			CENTRAL SUPPORT	END SPAN l_y			
	K_x			$-S_x$	K_y			
	RATIO DEAD : LIVE LOAD			DEAD : LIVE LOAD ALL RATIOS	RATIO DEAD : LIVE LOAD			
	1 : 1	1 : 2	1 : 4		1 : 1	1 : 2	1 : 4	
2·00	14·5	13·9	13·5	8·20	*200*	*200*	*200*	
1·80		14·6		8·30		*162*		
1·60		16·0		8·49		122		
1·50		16·9		8·63		100		
1·40		18·1		8·83		80·9		
1·30		19·7		9·12		65·3		
1·20		21·9		9·54		52·3		
1·10		24·8		10·2		41·7		
1·00	29·2	29·0	28·9	11·2	33·9	33·0	32·3	
0·95		31·8		11·9		29·3		
0·90		35·2		12·9		26·0		
0·85		39·2		14·1		23·1		
0·80		44·6		15·8		20·5		
0·75		51·1		18·1		18·3		
0·70		60·0		21·3		16·3		
0·60		88·3		32·7		13·2		
0·50	147	149	151	59·2	11·1	11·0	11·0	

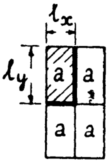
MOMENT ON SPAN $M_x = \frac{wl_x^2}{K_x}$

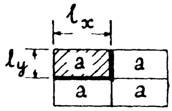
MOMENT ON SPAN $M_y = \frac{wl_y^2}{K_y}$

MOMENT AT SUPPORT $M_x = -\frac{wl_x^2}{S_x}$

TABLE 15A.—SLAB PANELS.

BENDING MOMENT FACTORS FOR DISTRIBUTED LOAD.





FOUR PANELS.

SHAPE OF PANEL	END SPAN l_x			CENTRAL SUPPORT	END SPAN l_y			CENTRAL SUPPORT
	K_x			$-S_x$	K_y			$-S_y$
$\frac{l_y}{l_x}$	RATIO DEAD : LIVE LOAD			DEAD : LIVE LOAD ALL RATIOS	RATIO DEAD : LIVE LOAD			DEAD : LIVE LOAD ALL RATIOS
	1 : 1	1 : 2	1 : 4		1 : 1	1 : 2	1 : 4	
2·00	14·7	14·1	13·6	8·50	*200*	*200*	*200*	136
1·80		15·1		8·76		159		92·0
1·60		16·6		9·22		109		60·4
1·50		17·7		9·58		89·8		48·5
1·40		19·2		10·1		73·8		38·7
1·30		21·2		10·8		60·5		30·8
1·20		23·9		11·8		49·6		24·6
1·10		27·7		13·4		40·6		19·7
1·00	34·1	33·2	32·5	16·0	34·1	33·2	32·5	16·0
0·95		36·9		17·8		30·1		14·5
0·90		41·6		20·2		27·2		13·2
0·85		47·1		23·3		24·7		12·2
0·80		54·6		27·5		22·4		11·3
0·75		64·7		33·3		20·4		10·5
0·70		78·1		41·3		18·7		9·92
0·60		123		69·7		16·0		9·04
0·50	*200*	*200*	*200*	136	14·7	14·1	13·6	8·50

MOMENT ON SPAN $M_x = \frac{wl_x^2}{K_x}$

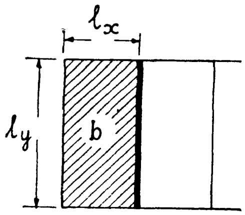
MOMENT ON SPAN $M_y = \frac{wl_y^2}{K_y}$

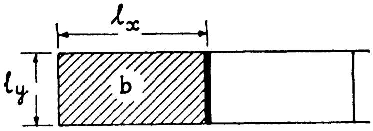
MOMENT AT SUPPORT $M_x = = \frac{wl_x^2}{S_x}$

MOMENT AT SUPPORT $M_y = - \frac{wl_y^2}{S_y}$

TABLE 16A.—SLAB PANELS.

BENDING MOMENT FACTORS FOR DISTRIBUTED LOAD.





SINGLE ROW—END PANEL.

SHAPE OF PANEL $\frac{l_y}{l_x}$	END SPAN l_x			FIRST SUPPORT	END SPAN l_y			
	K_x			$-S_x$	K_y			
	RATIO DEAD : LIVE LOAD			DEAD : LIVE LOAD ALL RATIOS	RATIO DEAD : LIVE LOAD			
	1 : 1	1 : 2	1 : 4		1 : 1	1 : 2	1 : 4	
2·00	14·5	13·9	13·5	10·1	*200*	*200*	*200*	
1·80		14·6		10·2		*162*		
1·60		16·0		10·3		122		
1·50		16·9		10·4		100		
1·40		18·1		10·5		80·9		
1·30		19·7		10·7		65·3		
1·20		21·9		10·9		52·3		
1·10		24·8		11·4		41·7		
1·00	29·2	29·0	28·9	12·0	33·9	33·0	32·3	
0·95		31·8		12·4		29·3		
0·90		35·2		13·0		26·0		
0·85		39·2		13·8		23·1		
0·80		44·6		14·9		20·5		
0·75		51·1		16·3		18·3		
0·70		60·0		18·3		16·3		
0·60		88·3		25·4		13·2		
0·50	147	149	151	42·0	11·1	11·0	11·0	

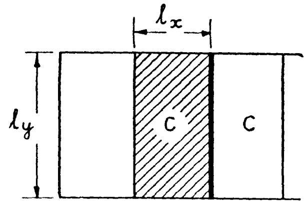
MOMENT ON SPAN $M_x = \frac{wl_x^2}{K_x}$

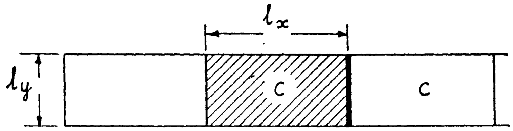
MOMENT ON SPAN $M_y = \frac{wl_y^2}{K_y}$

MOMENT AT SUPPORT $M_x = -\frac{wl_x^2}{S_x}$

TABLE 17A.—SLAB PANELS.

BENDING MOMENT FACTORS FOR DISTRIBUTED LOAD.





SINGLE ROW—INTERMEDIATE PANEL.

SHAPE OF PANEL	INTERIOR SPAN l_x			INTERMEDIATE SUPPORTS	SPAN l_y		
	K_x			$-S_x$	K_y		
$\frac{l_y}{l_x}$	RATIO DEAD : LIVE LOAD			DEAD : LIVE LOAD ALL RATIOS	RATIO DEAD : LIVE LOAD		
	1 : 1	1 : 2	1 : 4		1 : 1	1 : 2	1 : 4
2·00	19·1	17·5	16·4	12·1	*200*	*200*	*200*
1·80		18·5		12·2		*162*	
1·60		20·0		12·4		*128*	
1·50		21·0		12·5		*112*	
1·40		22·3		12·6		*98·0*	
1·30		24·0		12·8		84·7	
1·20		26·3		13·1		67·5	
1·10		29·3		13·6		53·1	
1·00	34·3	33·4	32·7	14·4	44·3	41·5	39·4
0·95		36·0		14·9		36·4	
0·90		39·3		15·6		31·9	
0·85		43·1		16·6		27·9	
0·80		48·1		17·9		24·3	
0·75		54·6		19·6		21·4	
0·70		63·0		22·0		18·7	
0·60		89·9		30·5		14·6	
0·50	142	146	147	50·4	11·9	11·8	11·6

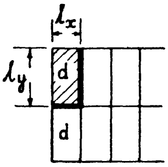
MOMENT ON SPAN $M_x = \frac{wl_x^2}{K_x}$

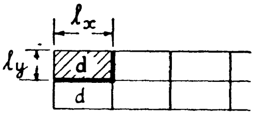
MOMENT ON SPAN $M_y = \frac{wl_y^2}{K_y}$

MOMENT AT SUPPORT $M_x = -\frac{wl_x^2}{S_x}$

TABLE 18A.—SLAB PANELS.

BENDING MOMENT FACTORS FOR DISTRIBUTED LOAD.





DOUBLE ROW—END PANEL.

SHAPE OF PANEL	END SPAN l_x			FIRST SUPPORT	END SPAN l_y			CENTRAL SUPPORT
	K_x			$-S_x$	K_y			$-S_y$
$\frac{l_y}{l_x}$	RATIO DEAD : LIVE LOAD			DEAD : LIVE LOAD ALL RATIOS	RATIO DEAD : LIVE LOAD			DEAD : LIVE LOAD ALL RATIOS
	1 : 1	1 : 2	1 : 4		1 : 1	1 : 2	1 : 4	
2·00	14·7	14·1	13·6	10·3	*200*	*200*	*200*	136
1·80		15·1		10·5		159		92·0
1·60		16·6		10·7		109		60·4
1·50		17·7		11·0		89·8		48·5
1·40		19·2		11·3		73·8		38·7
1·30		21·2		11·7		60·5		30·8
1·20		23·9		12·4		49·6		24·6
1·10		27·7		13·4		40·6		19·7
1·00	34·1	33·2	32·5	15·0	34·1	33·2	32·5	16·0
0·95		36·9		16·1		30·1		14·5
0·90		41·6		17·6		27·2		13·2
0·85		47·1		19·6		24·7		12·2
0·80		54·6		22·2		22·4		11·3
0·75		64·7		25·8		20·4		10·5
0·70		78·1		30·8		18·7		9·92
0·60		123		48·6		16·0		9·04
0·50	*200*	*200*	*200*	90·0	14·7	14·1	13·6	8·50

MOMENT ON SPAN $M_x = \frac{wl_x^2}{K_x}$

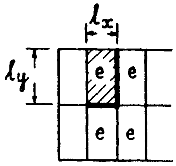
MOMENT ON SPAN $M_y = \frac{wl_y^2}{K_y}$

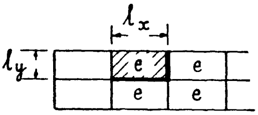
MOMENT AT SUPPORT $M_x = -\frac{wl_x^2}{S_x}$

MOMENT AT SUPPORT $M_y = -\frac{wl_y^2}{S_y}$

TABLE 19A.—SLAB PANELS.

BENDING MOMENT FACTORS FOR DISTRIBUTED LOAD.





DOUBLE ROW—INTERMEDIATE PANEL.

SHAPE OF PANEL	INTERIOR SPAN l_x			INTERIOR SUPPORT	SIDE SPAN l_y			CENTRAL SUPPORT
	K_x			$-S_x$	K_y			$-S_y$
$\frac{l_y}{l_x}$	RATIO DEAD : LIVE LOAD			DEAD : LIVE LOAD ALL RATIOS	RATIO DEAD : LIVE LOAD			DEAD : LIVE LOAD ALL RATIOS
	1 : 1	1 : 2	1 : 4		1 : 1	1 : 2	1 : 4	
2·00	19·2	17·6	16·5	12·4	*200*	*200*	*200*	*160*
1·80		18·7		12·6		*162*		*130*
1·60		20·4		12·9		*128*		*102*
1·50		21·5		13·2		*112*		89·0
1·40		23·0		13·5		95·0		69·4
1·30		25·0		14·1		76·8		53·7
1·20		27·7		14·9		61·8		41·2
1·10		31·4		16·1		49·5		31·4
1·00	38·3	36·7	35·5	18·0	41·7	39·5	37·8	24·0
0·95		40·2		19·4		35·2		21·0
0·90		44·6		21·1		31·4		18·5
0·85		49·7		23·5		28·0		16·3
0·80		56·7		26·6		25·0		14·5
0·75		66·0		30·9		22·5		13·0
0·70		78·3		37·0		20·3		11·8
0·60		120		58·3		16·8		10·1
0·50	*200*	*200*	*200*	108	15·2	14·5	14·0	8·99

MOMENT ON SPAN $M_x = \frac{wl_x^2}{K_x}$

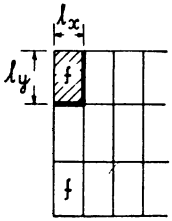
MOMENT ON SPAN $M_y = \frac{wl_y^2}{K_y}$

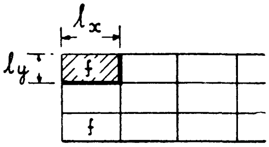
MOMENT AT SUPPORT $M_x = -\frac{wl_x^2}{S_x}$

MOMENT AT SUPPORT $M_y = -\frac{wl_y^2}{S_y}$

TABLE 20A.—SLAB PANELS.

BENDING MOMENT FACTORS FOR DISTRIBUTED LOAD.





CORNER PANEL.

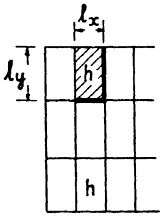
SHAPE OF PANEL $\frac{l_y}{l_x}$	END SPAN l_x			FIRST SUPPORT	END SPAN l_y			FIRST SUPPORT
	K_x			$-S_x$	K_y			$-S_y$
	RATIO DEAD : LIVE LOAD			DEAD : LIVE LOAD ALL RATIOS	RATIO DEAD : LIVE LOAD			DEAD : LIVE LOAD ALL RATIOS
	1 : 1	1 : 2	1 : 4		1 : 1	1 : 2	1 : 4	
2·00	14·7	14·1	13·6	10·3	*200*	*200*	*200*	90·0
1·80		15·1		10·5		159		64·6
1·60		16·6		10·7		109		42·7
1·50		17·7		11·0		89·8		35·3
1·40		19·2		11·3		73·8		29·2
1·30		21·2		11·7		60·5		24·2
1·20		23·9		12·4		49·6		20·5
1·10		27·7		13·4		40·6		17·3
1·00	34·1	33·2	32·5	15·0	34·1	33·2	32·5	15·0
0·95		36·9		16·1		30·1		14·1
0·90		41·6		17·6		27·2		13·3
0·85		47·1		19·6		24·7		12·7
0·80		54·6		22·2		22·4		12·1
0·75		64·7		25·8		20·4		11·6
0·70		78·0		30·8		18·7		11·2
0·60		123		48·6		16·0		10·6
0·50	*200*	*200*	*200*	90·0	14·7	14·1	13·6	10·3

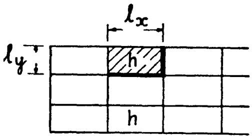
MOMENT ON SPAN $M_x = \frac{wl_x^2}{K_x}$ MOMENT ON SPAN $M_y = \frac{wl_y^2}{K_y}$

MOMENT AT SUPPORT $M_x = -\frac{wl_x^2}{S_x}$ MOMENT AT SUPPORT $M_y = -\frac{wl_y^2}{S_y}$

TABLE 21A.—SLAB PANELS.

BENDING MOMENT FACTORS FOR DISTRIBUTED LOAD.





EXTERIOR INTERMEDIATE PANEL.

SHAPE OF PANEL $\frac{l_y}{l_x}$	INTERMEDIATE SPAN l_x			INTERMEDIATE SUPPORT	END SPAN l_y			FIRST SUPPORT
	K_x			$-S_x$	K_y			$-S_y$
	RATIO DEAD : LIVE LOAD			DEAD : LIVE LOAD ALL RATIOS	RATIO DEAD : LIVE LOAD			DEAD : LIVE LOAD ALL RATIOS
	1 : 1	1 : 2	1 : 4		1 : 1	1 : 2	1 : 4	
2·00	19·2	17·6	16·5	12·4	*200*	*200*	*200*	*160*
1·80		18·7		12·6		*162*		115
1·60		20·4		12·9		*128*		75·5
1·50		21·5		13·2		*112*		60·6
1·40		23·0		13·5		95·0		48·4
1·30		25·0		14·1		76·8		38·6
1·20		27·7		14·9		61·8		30·7
1·10		31·4		16·1		49·5		24·6
1·00	38·3	36·7	35·5	18·0	41·7	39·5	37·8	20·0
0·95		40·2		19·4		35·2		18·1
0·90		44·6		21·1		31·4		16·5
0·85		49·7		23·5		28·0		15·2
0·80		56·7		26·6		25·0		14·1
0·75		66·0		30·9		22·5		13·2
0·70		78·3		37·0		20·3		12·4
0·60		120		58·3		16·8		11·3
0·50	*200*	*200*	*200*	108	15·2	14·5	14·0	10·6

MOMENT ON SPAN $M_x = \frac{wl_x^2}{K_x}$

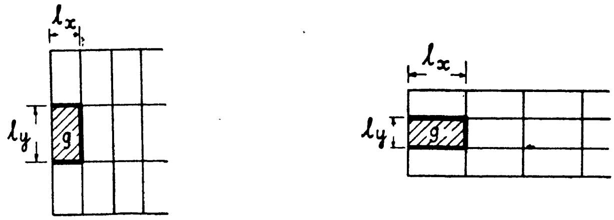
MOMENT ON SPAN $M_y = \frac{wl_y^2}{K_y}$

MOMENT AT SUPPORT $M_x = -\frac{wl_x^2}{S_x}$

MOMENT AT SUPPORT $M_y = -\frac{wl_y^2}{S_y}$

TABLE 22A.—SLAB PANELS.

BENDING MOMENT FACTORS FOR DISTRIBUTED LOAD.



CENTRAL END PANEL (THREE ROWS).

SHAPE OF PANEL	END SPAN l_x			FIRST SUPPORT	INTERMEDIATE SPAN l_y			FIRST SUPPORT
	K_x			$-S_x$	K_y			$-S_y$
$\frac{l_y}{l_x}$	RATIO DEAD : LIVE LOAD			DEAD : LIVE LOAD ALL RATIOS	RATIO DEAD : LIVE LOAD			DEAD : LIVE LOAD ALL RATIOS
	1 : 1	1 : 2	1 : 4		1 : 1	1 : 2	1 : 4	
2·00	15·2	14·5	14·0	10·6	*200*	*200*	*200*	90·0
1·80		15·6		10·9		152		62·5
1·60		17·6		11·5		107		42·7
1·50		18·9		12·0		89·0		35·3
1·40		20·9		12·6		74·7		29·2
1·30		23·5		13·5		62·2		24·2
1·20		26·9		14·8		52·4		20·5
1·10		32·1		16·8		43·6		17·3
1·00	41·7	39·5	37·8	20·0	38·3	36·7	35·3	15·0
0·95		44·5		22·3		33·9		14·1
0·90		50·9		25·2		31·0		13·3
0·85		58·2		29·1		28·6		12·7
0·80		68·9		34·4		26·2		12·1
0·75		82·4		41·6		24·3		11·6
0·70		101		51·6		22·6		11·2
0·60		*139*		87·2		19·7		10·6
0·50	*200*	*200*	*200*	*160*	19·2	17·6	16·5	10·3

MOMENT ON SPAN $M_x = \frac{wl_x^2}{K_x}$

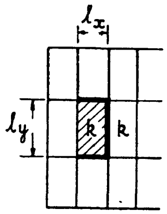
MOMENT ON SPAN $M_y = \frac{wl_y^2}{K_y}$

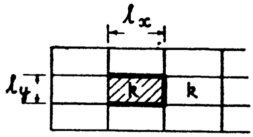
MOMENT AT SUPPORT $M_x = -\frac{wl_x^2}{S_x}$

MOMENT AT SUPPORT $M_y = -\frac{wl_y^2}{S_y}$

TABLE 23A.—SLAB PANELS.

BENDING MOMENT FACTORS FOR DISTRIBUTED LOAD.





INTERIOR PANEL (THREE ROWS).

SHAPE OF PANEL	INTERIOR SPAN l_x			INTERMEDIATE SUPPORT	INTERIOR SPAN l_y			FIRST SUPPORT
	K_x			$-S_x$	K_y			$-S_y$
$\frac{l_y}{l_x}$	RATIO DEAD : LIVE LOAD			DEAD : LIVE LOAD ALL RATIOS	RATIO DEAD : LIVE LOAD			DEAD : LIVE LOAD ALL RATIOS
	1 : 1	1 : 2	1 : 4		1 : 1	1 : 2	1 : 4	
2·00	19·5	17·9	16·7	12·7	*200*	*200*	*200*	*160*
1·80		19·0		13·1		*162*		115
1·60		20·9		13·8		*128*		75·5
1·50		22·3		14·4		112		60·6
1·40		24·1		15·1		92·6		48·4
1·30		26·5		16·2		75·8		38·6
1·20		29·9		17·8		62·0		30·7
1·10		34·6		20·2		51·7		24·6
1·00	44·3	41·5	39·4	24·0	44·3	41·5	39·4	20·0
0·95		46·1		26·7		37·5		18·1
0·90		52·0		30·3		34·0		16·5
0·85		58·9		35·0		30·9		15·2
0·80		68·3		41·3		28·0		14·1
0·75		81·1		49·9		25·6		13·2
0·70		98·0		62·0		23·5		12·4
0·60		*139*		104		20·2		11·3
0·50	*200*	*200*	*200*	*160*	19·5	17·9	16·7	10·6

MOMENT ON SPAN $M_x = \frac{wl_x^2}{K_x}$

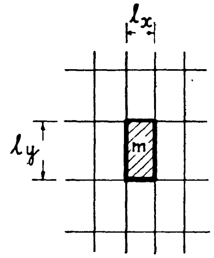
MOMENT ON SPAN $M_y = \frac{wl_y^2}{K_y}$

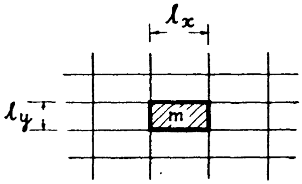
MOMENT AT SUPPORT $M_x = -\frac{wl_x^2}{S_x}$

MOMENT AT SUPPORT $M_y = -\frac{wl_y^2}{S_y}$

TABLE 24A.—SLAB PANELS.

BENDING MOMENT FACTORS FOR DISTRIBUTED LOAD.





FULLY CONTINUOUS PANEL.

SHAPE OF PANEL	INTERIOR SPAN l_x			INTERIOR SUPPORTS	INTERIOR SPAN l_y			INTERIOR SUPPORTS
	K_x			$-S_x$	K_y			$-S_y$
$\frac{l_y}{l_x}$	RATIO DEAD : LIVE LOAD			DEAD : LIVE LOAD ALL RATIOS	RATIO DEAD : LIVE LOAD			DEAD : LIVE LOAD ALL RATIOS
	1 : 1	1 : 2	1 : 4		1 : 1	1 : 2	1 : 4	
2·00	19·5	17·9	16·7	12·7	*200*	*200*	*200*	*160*
1·80		19·0		13·1		*162*		*130*
1·60		20·9		13·8		*128*		90·6
1·50		22·3		14·4		112		72·8
1·40		24·1		15·1		92·6		58·1
1·30		26·5		16·2		75·8		46·3
1·20		29·9		17·8		62·0		36·9
1·10		34·6		20·2		50·7		29·6
1·00	44·3	41·5	39·4	24·0	44·3	41·5	39·4	24·0
0·95		46·1		26·7		37·5		21·8
0·90		52·0		30·3		34·0		19·9
0·85		58·9		35·0		30·9		18·2
0·80		68·3		41·3		28·0		16·9
0·75		81·1		49·9		25·6		15·8
0·70		98·0		62·0		23·5		14·9
0·60		*139*		104		20·2		13·5
0·50	*200*	*200*	*200*	*160*	19·5	17·9	16·7	12·7

MOMENT ON SPAN $M_x = \frac{wl_x^2}{K_x}$

MOMENT ON SPAN $M_y = \frac{wl_y^2}{K_y}$

MOMENT AT SUPPORT $M_x = -\frac{wl_x^2}{S_x}$

MOMENT AT SUPPORT $M_y = -\frac{wl_y^2}{S_y}$

(iv) *Loads and bending moments in supporting beams.*

(1) The loads on the supporting beams for a two-way rectangular slab may be assumed as the uniformly distributed load within the tributary areas of the slab bounded by the intersection of 45 degree lines from the corners with the median line of the slab parallel to the long side, see Fig. 4. On this basis the total loads on the short and long span beams due to one loaded slab are given by the following equations :

Load on short span beam,

$$\text{i.e. } W_s = \frac{wl_x^2}{4} \quad \ldots\ldots\ldots\ldots \quad (5)$$

Load on long span beam,

$$\text{i.e. } W_l = \frac{wl_xl_y}{2} - W_s \quad \ldots\ldots\ldots \quad (6)$$

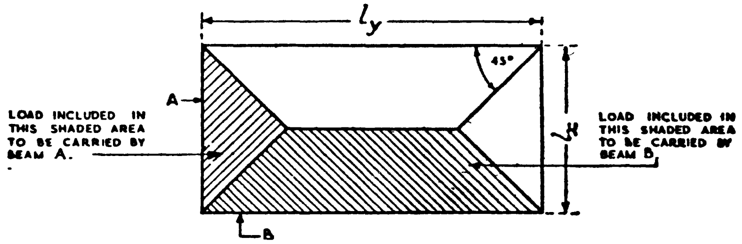

l_y
45°
A
LOAD INCLUDED IN THIS SHADED AREA TO BE CARRIED BY BEAM A.
l_x
LOAD INCLUDED IN THIS SHADED AREA TO BE CARRIED BY BEAM B
B

FIG. 4.—DIAGRAM SHOWING THE LOAD CARRIED BY SUPPORTING BEAMS.

(2) The bending moments in the beams may be obtained for the load distribution assumed in (1) above or may be determined by assuming that the loading is equivalent to a uniform load per unit length of the beam of the following amounts :

$$\text{In the short span } \frac{wl_x}{3}$$

$$\text{In the long span } \frac{wl_x}{6}\left[3 - \left(\frac{l_x}{l_y}\right)^2\right]$$

It is sometimes specified that the distribution of load to the supports should be the same as the load distribution derived from the Grashof-Rankine method (see *Table* 13A). This method, however, leads in the case of long panels to an extremely small proportion of the load being carried on the shorter supports. The geometrical method of estimating the loads carried by the beams given by Clause 309 (*c*) (iv) has not this disadvantage, and has been commonly used in this country.

310. Resistance to shear.

(*a*) *General.*

(i) The shear stress, s, at any cross-section in a reinforced concrete beam or slab should be calculated from the following equation :

$$s = \frac{S}{ba} \quad \ldots\ldots\ldots\ldots \quad (7)$$

where S is the total shearing force across the section.

b is the breadth of a rectangular beam or the breadth of the rib of a T- or L-beam, and

a is the arm of the resistance moment.

(ii) Where at any cross-section the shear stress, as calculated from equation (7) above, exceeds the permissible shear stress for the concrete, the whole shearing force should be provided for by the tensile resistance of the shear reinforcement acting in proper combination with the compression in the concrete. Moreover, even with the whole shearing force so provided for, the shearing stress as calculated from equation (7) above should not exceed four times the permissible shear stress for the concrete alone.

Calculations of shear stresses in the case of solid floor slabs are usually unnecessary. Such calculations are, in the majority of structures, required only for beams and for certain types of column bases. For beams of moderate spans and normal loading, calculations of shear stress are only made for the points of maximum shear, that is adjacent to the supports.

It is customary to assume that the maximum shearing force (S) at these sections is equal to the maximum reaction on the column or other support imposed by the member in question. For continuous beams with large spans, where an envelope of maximum bending moments is drawn, the maximum shearing force to be assumed at any section can usually be obtained from the slope of the curve at a point immediately above or below the section. Equation (7) strictly applies to beams of uniform depth. In members of varying depth (*Fig.* 5A) the equation should be modified to read

$$s = \frac{S \pm \dfrac{M}{d}\tan\alpha}{ba},$$

the negative sign applying where the bending moment (M) increases numerically in the same direction as the effective depth (d) increases, and the positive sign where the moment decreases numerically in this direction.

The relationship between the requirements for shear and bond should be noted, since, in some cases, the maximum shearing force that a section can resist is limited by considerations of bond. Thus, from considerations of shear (equation 7), $\frac{S}{a} = sb$, and from considerations of bond (Clause 307) the maximum shear is limited to $\frac{S}{a} = B_2 o$, where B_2 is the permissible *local* bond stress.

If the bars are all of one size and are placed in a single layer, and if the number of bars is N and the clear distances between the bars and the cover are each equal to the diameter of the bars, as required by Clauses 304 and 305, $s = \frac{N\pi}{2N + 1} . B_2$. As N increases, this tends to the limiting value $s = 1{\cdot}57B_2$. If s_c is the permissible shearing stress for plain concrete, $\frac{s}{s_c} = 1{\cdot}57\frac{B_2}{s_c}$.

The ratio of permissible local bond stress to permissible shearing stress on plain concrete varies from 1·8 for 1 : 2 : 4 Portland cement concrete to 1·7 for 1 : 1 : 2 concrete (and is 1·7 for 1 : 2 : 4 high-alumina cement concrete). For a single layer of bars, therefore, the ratio $\frac{s}{s_c}$ has limiting values between 2·6 and 2·8, that is well below the maximum permissible value of 4. For practical numbers of bars the ratio will be still less. Thus for bars with the spacing and cover equal to the diameter of the bar the extreme values are

Number of bars .	1	2	3	4	6
1 : 2 : 4 concrete .	1·9	2·3	2·4	2·5	2·6
1 : 1 : 2 concrete .	1·8	2·1	2·3	2·4	2·5

Where either the spacing or the cover, or both, is greater than the diameter of the bar the values of $\frac{s}{s_c}$ will be less than those tabulated. Where, however, more than one layer of bars is used, the values will be greater and may exceed the maximum permissible value of 4. Where the intensity of shear exceeds three times the stress permitted in the plain concrete, particular care should be taken both with regard to the provision of shear reinforcement and the adequate anchorage of the tensile bars.

Shear Stresses in Column Bases and Footings.—The critical section for shear stresses in column bases is not specifically mentioned in the Code. The method given for calculating the shearing stresses at column heads in flat slabs (Clause 322) is, however, applicable. The shear stress should be calculated on a vertical section at a distance equal to the effective depth of the base from the face of the column or other compression member.

> (*b*) *Shear reinforcement.*
>
> (i) A stirrup in reinforced concrete should pass round, or be otherwise adequately secured to, the appropriate tensile reinforcement; and such stirrup should have both its ends anchored properly.
>
> (ii) Tensile reinforcement which is inclined and carried through a depth of beam equal to the arm of the resistance moment will also act as shear reinforcement provided it is anchored sufficiently.
>
> (iii) Where two or more types of shear reinforcement are used in conjunction, the total shearing resistance of the beam may be assumed to be the sum of the shearing resistances computed for each type separately.
>
> (iv) The spacing of stirrups should not exceed a distance equal to the arm of the resistance moment. The resistance to shear, S, should then be calculated from the following equation :
>
> $$S = \frac{t_w A_w a}{p} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$
>
> where t_w is the permissible tensile stress in the shear reinforcement.
> A_w is the cross-sectional area of the stirrup.
> a is the arm of the resistance moment.
> p is the pitch or spacing of stirrups.

The essential need in reference to the connection of the stirrups with the tensile bars is that the strands of each stirrup shall pass around one or more of the bars forming the tensile reinforcement, although the number of bars so embraced may comprise only a fraction of the total tensile reinforcement.

Further, it is not essential in order effectively to anchor the stirrups or ties at both ends that each strand of the stirrups should embrace a compression bar or bars. It is merely necessary that the ends of the stirrups or ties should be anchored within the compressive zone so that the maximum tensile stress can be developed in the various strands. This anchorage may consist of a sufficient length of stirrup turned horizontally or otherwise bent so as to prevent slip.

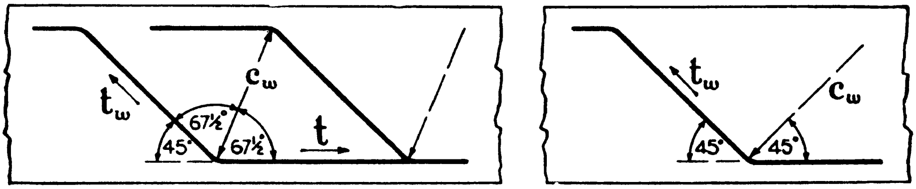
> (v) The resistance to shear at any section of a beam, reinforced with inclined bars, may be calculated on the assumption that the inclined bars form the tension members of one or more single systems of lattice girders in which the concrete forms the compression members. The shear resistance at any vertical section should then be taken as the sum of the vertical components of the tension and compression forces cut by the section. Care must be taken that such assumptions do not involve greater stresses in the horizontal bars than the permissible stresses.

When bottom tensile bars are bent up to resist shearing forces in a continuous beam it is customary to continue them over the support so that they may also act as top tensile reinforcement. For such bars the required anchorage [Clause 310 (*b*) (ii)] is automatically provided.

It will be noted that the method of treating bent-up bars as tensile members of a lattice girder is not obligatory. It will also be found that the lattice-girder method of design leads to a more conservative estimate of shear resistance than other methods in use.

With a series of bent-up bars it is quite safe and is common practice to assume that the compressive forces act at an angle of 45 deg. to the axis of the beam and to neglect any theoretically excessive tensile stress induced in the horizontal portion of the bar.

Consider a single system for each method and assume the bars to be bent up at 45 deg. as in *Fig.* 12A. With the lattice-girder method it is clear that to keep t_w and t equal, c_w must bisect the angle between them. If c_w is inclined nearer to t then t becomes greater than t_w, which consequently can never be allowed to reach the maximum tensile stress permitted.



LATTICE GIRDER METHOD. ALTERNATIVE METHOD.

FIG. 12A.

This is essentially the restriction contained in the final sentence of Clause 310 (*b*) (v), from which it follows that the most economical spacing of bars bent up at 45 deg. is that shown in *Fig.* 12A. The same applies, of course, to angles other than 45 deg., that is for equal tension in the horizontal and inclined portions of bars the compressive stress or force must bisect the angle between them.

With the alternative method, c_w is considered to act at 45 deg. (instead of 67½ deg.) with a correspondingly wider spacing of bent-up bars.

It should be appreciated that all methods of dealing with bent-up bars are open to objection insofar as it is necessary to consider the beam as acting, for the purpose of resistance to shearing forces, as some other type of construction.

COLUMNS

311. General.

(*a*) LONGITUDINAL REINFORCEMENT.

A reinforced concrete column should have longitudinal steel reinforcement, and the cross-sectional area of such reinforcement should not be less than 0·8 per cent nor more than 8 per cent of the gross cross-sectional area of the column required to transmit all the loading in accordance with this code.

A reinforced concrete column having helical reinforcement should have at least six bars of longitudinal reinforcement within this helical reinforcement. The longitudinal bars should be in contact with the helical reinforcement and equidistant around its inner circumference.

At a splice in a longitudinal reinforcement, the spliced bars should overlap longitudinally through a distance of not less than 24 times the diameter of the smaller bar.

The bars should not be less than ½ in. in diameter.

In determining a suitable cross section of a column, the following requirements should be observed in addition to those given in Clause 311 (*a*). The minimum permissible lateral spacing of main longitudinal bars is two diameters (Clause 305) ; the minimum cover of concrete to the main bars is 1½ in. or the diameter of the bar, whichever is the greater [Clause 304 (*b*)] ; the minimum cover of concrete to the transverse reinforcement is ½ in. or the diameter of such reinforcement, whichever is the greater [Clause 304 (*e*)].

(*b*) TRANSVERSE REINFORCEMENT.

(i) *General.* A reinforced concrete column should have transverse or helical reinforcement so disposed as to provide all necessary restraint against the buckling of each of the longitudinal reinforcements. Every bar in a column near the face should be properly linked. The ends of such transverse reinforcement should be properly anchored.

(ii) *Pitch.* The pitch of transverse reinforcement should be not more than the least of the three following distances:

(1) the least lateral dimension of the column ;
(2) twelve times the diameter of the smallest longitudinal reinforcement in the column ;
(3) 12 in.

(iii) *Helical reinforcement.* Helical reinforcement should be of regular formation, with the turns of the helix spaced evenly, and its ends should be anchored properly. Where an increased load on the column on account of the helical reinforcement is allowed for under clause 312 (*a*) (iii) the pitch of the helical turns should be not more than 3 in., nor more than one-sixth of the core diameter of the column, nor less than 1 in. nor less than three times the diameter of the steel bar forming the helix. In other cases the requirements of paragraph (ii) above should be complied with.

(iv) ***Diameter.*** **The diameter of the transverse reinforcements should not be less than one quarter the diameter of the main rods, and in no case less than $\frac{3}{16}$ in.**

At floors and similar positions where columns are afforded lateral support there is usually a considerable congestion of reinforcement caused by beams entering the column from several directions. There is also an increased mass of concrete at these points, and where the column is embraced on all sides the necessity for lateral reinforcement is greatly reduced if not entirely eliminated. It is therefore safe to reduce, and sometimes to omit, the transverse reinforcement over the lengths of column in question, with the advantage that concreting is facilitated and dense concrete more easily obtained.

Transverse reinforcement of columns is of two kinds, namely, (*a*) lateral ties, and (*b*) helical reinforcement. The purpose of lateral ties is primarily to restrain the longitudinal bars from outward buckling. The diameter and pitch of the ties are therefore related to the diameter of the longitudinal bars, with further restrictions in regard to minimum diameter ($\frac{3}{16}$ in.) and to spacing relative to the dimensions of the column. *Table* 25A gives the minimum permissible diameters and maximum spacing of lateral ties for the various sizes of longitudinal bars commonly employed.

TABLE 25A.—LATERAL TIES.

Diameter of longitudinal bar (in.)	Minimum diameter of ties (in.) (One-quarter the diameter of the longitudinal bar but not less than $\frac{3}{16}$ in.)	Maximum pitch of ties (in.)
$\frac{1}{2}$	$\frac{3}{16}$	6
$\frac{5}{8}$	$\frac{3}{16}$	7$\frac{1}{2}$
$\frac{3}{4}$	$\frac{3}{16}$	9
$\frac{7}{8}$	$\frac{1}{4}$	10$\frac{1}{2}$
1	$\frac{1}{4}$	12
1$\frac{1}{8}$	$\frac{5}{16}$	
1$\frac{1}{4}$	$\frac{5}{16}$	
1$\frac{3}{8}$	$\frac{3}{8}$	
1$\frac{1}{2}$	$\frac{3}{8}$	
1$\frac{3}{4}$	$\frac{7}{16}$	
2	$\frac{1}{2}$	

Note.—The figures in this table are subject to the provision that the pitch is not more than the smallest side of the column.

The restriction concerning the spacing of lateral ties in terms of the minimum lateral dimension of the column has special significance in columns with re-entrant angles. For example, in an L-shape column, with the two long sides 12 in. and the remainder 6 in., a maximum pitch of 6 in. would be required whatever the size of reinforcement used.

The action of helical reinforcement is to restrain the concrete core from lateral expansion, thereby increasing the maximum load it will carry before collapse. An upper limit to the spacing of the coils is required in order to ensure that the helical reinforcement properly restrains the concrete core and maintains a contribution to the strength of the column proportional to its volume.

The requirement concerning the minimum pitch ensures a sufficient space between the coils to permit the formation of the cover without honeycombing.

A further restriction affecting the helical reinforcement is contained in the requirement, following equation (10), that the allowable load on the concrete core within the helical reinforcement shall not exceed the load that would be carried by the gross cross-sectional area of the concrete when stressed to one-half of the specified works-cube strength. This requirement ensures that the lateral movements in helically-reinforced columns are kept within limits necessary to prevent the possibility of spalling of the cover. In effect it limits the amount of helical reinforcement that can usefully be employed.

The minimum diameter permissible for helical reinforcement is $\frac{3}{16}$ in., while the maximum diameter that can be employed is governed by the requirement that the minimum pitch shall be three times the diameter of the helical bar and that such pitch shall not exceed 3 in. The maximum diameter of helical bar permissible is therefore 1 in.

In considering the question of large diameter helical bars, practical considerations such as the winding of the coils and the length of bars obtainable must be taken into account. For example, in a 36-in. diameter column, 1-in. diameter bars 45 ft. long will provide approximately one linear foot of helical coil, about 20 per cent. of each of these lengths being required for splicing purposes. It may be useful in this connection to state that helical reinforcement up to $\frac{7}{8}$ in. diameter has been successfully employed in this country.

The cover of concrete required for helical reinforcement is $\frac{1}{2}$ in. or the diameter of the helical bar, whichever is greater.

312. Permissible loads.

(*a*) AXIALLY LOADED COLUMNS.

(i) *Short columns defined.* Columns may be treated as short columns where the ratio of the effective column length to least lateral dimension does not exceed 15. The maximum permissible stresses for these should be as specified in clause 303 (*a*) and (*b*).

The permissible stresses in the concrete and steel for use in axially-loaded columns are as follows:

Nominal mix	Direct stress in concrete (c) lb. per sq. in.	
	Normal grade	Lower grade
1 : 1 : 2	1140	855
1 : 1$\frac{1}{2}$: 3	950	712
1 : 2 : 4	760	570

These figures are for Portland cement concrete complying with the requirements of *Table* 1. The figure for the 1 : 1 : 2 mix of Normal Grade is applicable also to 1 : 2 : 4 high-alumina cement concrete complying with the requirements of *Table* 2.

The crushing strength required from the works test (u_w) is 3·95 times the permissible direct stress in the concrete (c).

Type of steel	Stress (c_1) for longitudinal steel in direct compression (lb. per sq. in.)	Tensile stress (t_b) in helical reinforcement (lb. per sq. in.)
Mild steel (B.S. 785)	18,000	13,500
Mild steel with guaranteed yield point; or medium tensile steel.	Half the guaranteed yield point, but not more than 20,000.	15,000
High-tensile steel (B.S. 785 or B.S. 1144).	20,000	18,000

(ii) *Short columns with lateral ties.* The axial load, P, permissible on a short column reinforced with longitudinal bars and lateral ties should not exceed that given by the following equation :

$$P = cA + c_1A_c \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

where c is the permissible stress for the concrete in direct compression.

A is the cross-sectional area of concrete excluding any finishing material and reinforcing steel.

c_1 is the permissible compression stress for column bars.

A_c is the cross-sectional area of the longitudinal steel.

Equation (9) is based on tests which have shown that failure of a column occurs when the concrete and the steel both reach their maximum load-carrying capacities. The strength of the concrete in a column has been shown to be equal to about two-thirds of the crushing strength of a cube, and the maximum load-carrying capacity of the reinforcement is reached at its yield point. Thus the load at failure of a column is approximately (two-thirds times the concrete cube strength times the area of concrete) plus (the yield point stress of steel times the area of bars), which, when the concrete has the exact strength demanded from the works tests and mild or medium-tensile steel is used, is equal to $2{\cdot}63cA + 2c_1A_c$.

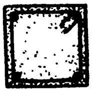
The minimum load factor with this type of column is obtained by using concrete of minimum strength reinforced with the maximum percentage of steel, and is approximately 2·2. The maximum load factor obtainable is nearly 2·6 and occurs when concrete of maximum strength is used with the minimum allowable percentage of steel. Since the strength of concrete will usually be greater than the minimum value specified for a particular concrete, the actual load factor will be higher than the foregoing values.

The gross area of the smallest size of column with lateral ties for any given load (W lb.) is approximately $\frac{W}{2650}$ sq. in., for which the strongest concrete and the maximum percentage of steel at the higher allowable stress would be required. In practice, a somewhat larger column might be necessary to satisfy the requirements for spacing of the bars and for adequate cover of concrete.

In *Tables* 26A and 27A, which give the permissible loads on various sizes of short axially-loaded square columns, three loads have been computed for each size of column ; the first with an arrangement of longitudinal bars approxi-

TABLE 26A.

SQUARE COLUMNS



PERMISSIBLE LOAD (LB.)

Size	Longitudinal steel			Load carried by longitudinal bars at 18,000 lb. per sq. in.	NORMAL-GRADE CONCRETE			LOWER-GRADE CONCRETE		
(in.)	Bars No.	Dia.	Area (sq. in.)		1 : 2 : 4	1 : 1½ : 3	1 : 1 : 2	1 : 2 : 4	1 : 1½ : 3	1 : 1 : 2
					760	950	1140	570	712·5	855
8	4	½	0·79	14,100	62,200	74,200	86,200	50,200	59,200	68,200
	4	1	3·14	56,600	102,800	114,400	125,900	91,200	99,900	108,600
	4	1¼	4·91	88,400	133,300	144,500	155,700	122,000	130,500	138,900
9	4	½	0·79	14,100	75,100	90,300	105,600	59,900	71,300	82,700
	4	1	3·14	56,600	115,700	130,500	145,300	100,900	112,000	123,100
	8	1	6·28	113,100	169,900	184,100	198,300	155,700	166,300	177,000
10	4	⅝	1·23	22,100	97,200	115,900	134,700	78,400	92,500	106,500
	4	1⅛	3·98	71,600	144,500	162,800	181,000	126,300	140,000	153,700
	8	1⅛	7·95	143,100	213,100	230,600	248,100	195,600	208,700	221,800
11	4	⅝	1·23	22,100	113,100	135,900	158,600	90,400	107,400	124,500
	4	1¼	4·91	88,400	176,600	198,600	220,700	154,500	171,100	187,600
	4	1¾	9·62	173,200	257,800	279,000	300,200	236,700	252,600	268,400
12	4	⅝	1·23	22,100	130,600	157,700	184,900	103,500	123,800	144,200
	4	1⅜	5·94	106,900	211,800	238,100	264,300	185,600	205,300	225,000
	8	1¼	9·82	176,700	278,700	304,200	329,700	253,200	272,300	291,400
13	4	¾	1·77	31,800	158,900	190,700	222,400	127,100	151,000	174,800
	8	1	6·28	113,100	236,800	267,700	298,600	205,900	229,000	252,200
	10	1¼	12·27	220,900	340,000	369,800	399,600	310,200	332,600	354,900
14	4	¾	1·77	31,800	179,400	216,300	253,200	142,500	170,200	197,900
	8	1	6·28	113,100	257,300	293,300	329,400	221,200	248,300	275,300
	8	1½	14·14	254,500	392,700	427,200	461,800	358,200	384,100	410,000
15	6	⅝	1·84	33,100	202,700	245,100	287,500	160,300	192,100	223,900
	8	1¼	9·82	176,700	340,200	381,100	422,000	299,400	330,000	360,700
	12	1⅜	17·82	320,700	478,200	517,500	556,900	438,800	468,300	497,800
16	4	⅞	2·41	43,300	236,000	284,200	332,400	187,800	224,000	260,100
	8	1⅜	11·88	213,800	399,300	445,700	492,100	352,900	387,700	422,500
	8	1¾	19·24	346,400	526,300	571,300	616,300	481,400	515,100	548,800
17	4	⅞	2·41	43,300	261,100	315,600	370,000	206,700	247,500	288,300
	8	1⅜	11·88	213,800	424,400	477,100	529,700	371,800	411,200	450,700
	12	1½	21·21	381,700	585,200	636,100	687,000	534,300	572,500	610,700
18	4	1	3·14	56,600	300,500	361,400	422,400	239,500	285,200	330,900
	8	1½	14·14	254,500	490,000	548,900	607,700	431,100	475,300	519,400
	8	2	25·13	452,400	679,500	736,300	793,100	622,800	665,300	707,900

TABLE 27A.

SQUARE COLUMNS

PERMISSIBLE LOAD (LB.)

Size	Longitudinal steel			Load carried by longitudinal bars at 18,000 lb. per sq. in.	NORMAL-GRADE CONCRETE			LOWER-GRADE CONCRETE		
(in.)	Bars No.	Dia.	Area (sq. in.)		1 : 2 : 4	1 : 1½ : 3	1 : 1 : 2	1 : 2 : 4	1 : 1½ : 3	1 : 1 : 2
					760	950	1140	570	712·5	855
19	4	1	3·14	56,600	328,500	396,500	464,500	260,500	311,500	362,500
	12	1¼	14·73	265,100	528,300	594,000	659,800	462,500	511,800	561,200
	12	1¾	28·86	519,500	772,000	835,000	898,200	708,800	756,200	803,500
20	8	¾	3·53	63,600	364,900	440,200	515,600	289,600	346,100	402,600
	12	1⅜	17·82	320,700	611,200	683,800	756,400	538,500	593,000	647,500
	10	2	31·42	565,500	845,600	915,700	985,700	775,600	828,100	880,600
21	8	¾	3·53	63,600	396,100	479,200	562,300	313,000	375,300	437,600
	8	1¾	19·24	346,400	666,900	747,000	827,200	586,800	646,900	707,000
	10	2	31·42	565,500	876,800	954,600	1,032,000	799,000	857,300	915,700
22	8	⅞	4·81	86,600	450,800	541,800	632,900	359,700	428,000	496,300
	12	1½	21·21	381,700	733,400	821,400	909,300	645,500	711,400	777,400
	12	2	37·70	678,600	1,018,000	1,103,000	1,187,000	933,000	996,600	1,060,000
24	8	⅞	4·81	86,600	520,700	629,200	737,800	412,200	493,600	575,000
	12	1¾	28·86	519,500	935,400	1,039,000	1,143,000	831,400	909,400	987,300
	18	1¾	43·30	779,300	1,184,000	1,285,000	1,387,000	1,083,000	1,159,000	1,235,000
26	8	1	6·28	113,100	622,100	749,300	876,600	494,800	590,300	685,700
	12	1¾	28·86	519,500	1,011,000	1,134,000	1,257,000	888,400	980,600	1,073,000
	16	2	50·27	904,800	1,380,000	1,499,000	1,618,000	1,261,000	1,351,000	1,440,000
28	8	1	6·28	113,100	704,200	851,900	999,700	556,400	667,200	778,000
	12	2	37·70	678,600	1,246,000	1,388,000	1,529,000	1,104,000	1,210,000	1,317,000
	20	2	62·83	1,131,000	1,679,000	1,816,000	1,953,000	1,542,000	1,645,000	1,748,000
30	12	⅞	7·22	129,900	808,400	978,000	1,148,000	638,800	766,000	893,200
	16	1¾	38·48	692,700	1,348,000	1,511,000	1,675,000	1,184,000	1,307,000	1,429,000
	22	2	69·12	1,244,000	1,876,000	2,033,000	2,191,000	1,718,000	1,836,000	1,954,000
32	12	1	9·43	169,600	940,700	1,133,000	1,326,000	748,000	892,500	1,037,000
	16	2	50·27	904,800	1,645,000	1,830,000	2,015,000	1,460,000	1,599,000	1,737,000
	24	2	75·40	1,357,000	2,078,000	2,258,000	2,439,000	1,898,000	2,033,000	2,168,000
34	8	1¼	9·82	176,700	1,048,000	1,266,000	1,483,000	830,000	993,400	1,157,000
	16	2	50·27	904,800	1,745,000	1,955,000	2,165,000	1,535,000	1,693,000	1,850,000
	28	2	87·96	1,583,000	2,395,000	2,598,000	2,801,000	2,192,000	2,344,000	2,497,000
36	8	1⅜	11·88	213,800	1,190,000	1,434,000	1,678,000	945,800	1,129,000	1,312,000
	32	1½	56·55	1,018,000	1,960,000	2,195,000	2,431,000	1,724,000	1,901,000	2,078,000
	28	2	87·96	1,583,000	2,501,000	2,731,000	2,960,000	2,272,000	2,444,000	2,616,000

mating to the minimum permissible percentage, the second with an intermediate quantity, and the last with a high percentage of longitudinal bars approaching the maximum permissible percentage.

It will be noted that for each case the longitudinal bars are all of one diameter, but alternative groupings of bars (having the same total area) can be substituted. Also the loads given are equally applicable to columns of other shapes provided that these have the same cross-sectional area and continue to satisfy the dimensional requirements for short columns.

(iii) *Short columns with helical reinforcement.* Where helical reinforcement is used, the axial load, P, permissible on a short column should not exceed that given by equations (9) and (10), whichever is the greater :

$$P = cA_k + c_1A_c + 2t_bA_b \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

where A_k is the cross-sectional area of concrete in the core, excluding the area of longitudinal reinforcements ;
t_b is the permissible stress in the helical reinforcement ; and
A_b is the equivalent area of helical reinforcement (volume of helix per unit length of the column).

The sum of the terms $cA_k + 2t_bA_b$ should not exceed $0{\cdot}5\ uA$, where u is the crushing strength of the concrete required from the works test.

Equation (10) is derived in the same way as equation (9). In this case, however, only the area of concrete within the core of the column (measured to the centre of the helical bar) is taken into account, and an additional term is included for the effect of the helical reinforcement. Tests have shown that the increase of strength due to the provision of helical bars is equivalent to the use of additional longitudinal steel with a volume per unit length of the column equal to twice that of the helical reinforcement. The full value of this reinforcement is not introduced in equation (10) since the permissible stress (t_b) for the helical bar is less than that (c_1) for the longitudinal steel.

The ultimate load, for a column in which the concrete is equal in strength to the works-cube requirements, and mild or medium-tensile steel is used, is thus approximately $2{\cdot}63cA_k + 2c_1A_c + 5{\cdot}4t_bA_b$. The load factor for helically-reinforced columns for these conditions is slightly higher than for columns with lateral ties having similar concrete and longitudinal steel, the range being approximately 2·3 to 2·6. The load factor increases a little as the amount of helical reinforcement is increased.

The smallest helically-reinforced column permissible is controlled by the requirements for the spacing of the helix and for the cover of concrete over the longitudinal and helical bars. With an octagonal column and eight longitudinal bars (placed at the corners to gain additional cover) the minimum size permissible is approximately 8 in. If eight $\frac{3}{4}$-in. longitudinal bars and a $\frac{5}{16}$-in. spiral at 1-in. pitch be employed, the maximum permissible load on this size of column would be about 65 tons.

In *Tables* 28A and 29A the permissible loads for helically-reinforced columns of various sizes are given. These columns are assumed to be short and axially loaded. For the purpose of deciding suitable limits for the amounts of longitudinal and helical reinforcement the columns have been assumed to be octagonal in section. The diameter of the core to be assumed depends upon the amount of cover required; in some cases this will be determined by the diameter of the helical bars and in others by the diameter of the longitudinal bars. Three loads are given for each size of column; the first in each case being that obtained by using a relatively small amount of reinforcement, the second is obtained by an intermediate quantity, and the last by a quantity of both helical and longitudinal bars approaching the maximum which is permissible, practicable, or useful. The maximum quantity of helical reinforcement that can usefully be used (having regard to the restriction on the average stress in the concrete on the gross area of the column) depends on the quality of the concrete. The amounts given in the table are useful for the whole range of strengths of concrete considered, and could be increased for the higher-strength concretes with a corresponding increase in permissible load of up to about 25 per cent.

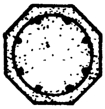
In considering the load-carrying capacity of columns with small percentages of helical reinforcement, it should be noted that, where the contribution of the latter is less than that of the concrete outside the core, a greater allowable load will be obtained by treating the columns as of the ordinary type and employing formula (9). The minimum volume of helical reinforcement required to contribute economically to the load-carrying capacity of a column depends on the stresses adopted for the concrete and the helix, as well as upon the amount of cover of concrete provided. In calculating the first of the three loads mentioned, the quantity of helical reinforcement adopted is just sufficient to ensure that the helical reinforcement increases the permissible load on the column for the whole range of strengths of concrete, with a stress in the helical reinforcement of 13,500 lb. per square inch. In the ordinary course of design it is unlikely that a column with helical reinforcement would be reinforced with very low percentages of reinforcement, but by giving the information in this way the reader is provided with practically the entire range of permissible loads for each size of column.

There is, of course, a variety of intermediate loads to be obtained for each size of column by suitably adapting the longitudinal and helical reinforcement between these limits.

> (iv) *Long columns.* The maximum permissible stresses in a reinforced concrete column or part thereof, having a ratio of effective column length to least lateral dimension between 15 and 45, should not exceed those which result from the multiplication of the appropriate maximum permissible stresses specified in clauses 303 (*a*) and (*b*) by the coefficient shown as appropriate for each ratio of effective column length to least lateral dimension in Table 11 [on page 81.]

TABLE 28A.

OCTAGONAL COLUMNS



PERMISSIBLE LOAD (LB.)

Size	Longitudinal reinforcement			Helical reinforcement			Load carried by longitudinal bars at 18,000 lb. per sq. in.	Load due to helical reinforcement at 13,500 lb. per sq. in.	NORMAL-GRADE CONCRETE			LOWER-GRADE CONCRETE		
	Bars		Area	Internal Dia.	Dia.	Pitch								
in.)	No.	Dia.	(sq. in.)	(in.)	(in.)	(in.)			1 : 2 : 4	1 : 1½ : 3	1 : 1 : 2	1 : 2 : 4	1 : 1½ : 3	1 : 1 : 2
	8	½	1·57	7½	5/16	1¼	28,300	40,600	104,200	113,000	121,800	95,400	102,000	108,600
10	8	¾	3·53	7½	5/16	1⅛	63,600	45,200	142,500	151,000	159,400	134,100	140,400	146,800
	8	1	6·28	7½	⅜	1⅞	113,100	53,700	199,000	207,100	215,100	190,900	197,000	203,000
	8	½	1·57	8½	5/16	1¼	28,300	45,900	119,300	130,600	141,900	108,000	116,500	125,000
11	8	⅞	4·81	8½	5/16	1	86,600	57,300	186,600	197,300	208,000	176,000	184,000	192,000
	8	1⅛	7·95	8½	⅜	1⅜	143,100	60,500	244,500	254,800	265,000	234,300	242,000	249,700
	8	½	1·57	9½	5/16	1¼	28,300	51,100	135,600	149,700	163,800	121,600	132,100	142,700
12	8	⅞	4·81	9½	5/16	1	86,600	63,800	204,200	217,700	231,100	190,800	200,900	211,000
	12	1	9·43	9	⅜	1¼	169,600	70,300	285,200	296,500	307,800	273,800	282,300	290,800
	8	½	1·57	10¾	5/16	1¼	28,300	57,600	157,700	175,600	193,600	139,700	153.200	166,700
13	8	1	6·28	10¾	5/16	1	113,100	72,000	253,300	270,400	287,400	236,300	249,000	261,900
	8	1¼	9·82	10¾	⅜	1¼	176,700	83,400	326,500	343,100	359,700	309,900	322,300	334,800
	8	½	1·57	11¾	5/16	1¼	28,300	62,800	176,600	198,000	219,400	155,200	171,300	187,300
14	8	1	6·28	11¾	⅜	1½	113,100	75,700	271,700	292,500	313,200	251,000	266,600	282,100
	13	1⅛	12·92	11	½	1⅞	232,600	102,200	403,900	421,200	438,400	386,600	399,600	412,500
	8	½	1·57	12¾	5/16	1¼	28,300	68,000	196,900	222,000	247,200	171,700	190,600	209,500
15	8	1⅛	7·95	12¾	⅜	1½	143,100	82,000	321,800	346,000	370,200	297,600	315,800	333,900
	12	1¼	14·73	12	½	1¾	265,100	119,000	466,100	486,700	507,200	445,600	461,000	476,400
	8	⅝	2·45	13¾	5/16	1¼	44,200	73,200	233,500	262,500	291,500	204,500	226,200	248,000
16	8	1¼	9·82	13¾	⅜	1½	176,700	88,200	376,600	404,500	432,400	348,700	369,600	390,500
	14	1¼	17·18	13	½	1¾	309,200	128,500	533,400	557,300	581,300	509,500	527,400	545,400
	8	⅝	2·45	15	5/16	1½	44,200	66,400	248,700	283,200	317,700	214,200	240,000	265,900
17	8	1¼	9·82	15	½	2¼	176,700	103,300	415,900	449,900	483,900	381,900	407,400	432,900
	8	1¾	19·24	14⅜	½	1¾	346,400	141,600	605,400	634,800	664,200	576,100	598,100	620,100
	8	⅝	2·45	16	5/16	1½	44,200	70,800	271,800	311,000	350,200	232,600	262,000	291,400
18	8	1⅜	11·88	16	½	2¼	213,800	122,100	489,400	527,800	566,200	451,100	479,800	508,600
	12	1½	21·21	15	⅝	2¼	381,700	180,700	692,000	724,400	756,800	659,600	683,900	708,200
	8	⅝	2·45	17	5/16	1½	44,200	75,100	296,300	340,500	384,800	252,000	285,200	318,400
19	8	1½	14·14	17	½	2¼	254,500	129,500	556,100	599,100	642,100	513,100	545,300	577,600
	13	1½	22·97	16	⅝	2¼	413,500	192,300	753,300	790,200	827,100	716,400	744,100	771,700

TABLE 29A.

OCTAGONAL COLUMNS

PERMISSIBLE LOAD (LB.)

Size	Longitudinal reinforcement			Helical reinforcement			Load carried by longitudinal bars at 18,000 lb. per sq. in.	Load due to helical reinforcement at 13,500 lb. per sq. in.	NORMAL-GRADE CONCRETE			LOWER-GRADE CONCRETE		
(in.)	Bars No.	Bars Dia.	Area (sq. in.)	Internal Dia. (in.)	Dia. (in.)	Pitch (in.)			1 : 2 : 4	1 : 1½ : 3	1 : 1 : 2	1 : 2 : 4	1 : 1½ : 3	1 : 1 : 2
	8	¾	3·53	18⅜	5/16	1¾	63,600	69,500	338,900	390,300	441,800	287,400	326,000	364,600
20	8	1½	14·14	18	½	2¼	254,500	136,900	585,000	633,400	681,700	536,600	572,900	609,200
	15	1½	26·51	17	⅝	2¼	477,100	203,800	846,200	887,500	928,900	804,900	835,900	866,900
	8	¾	3·53	19⅜	5/16	1¾	63,600	73,200	365,500	422,700	479,900	308,300	351,200	394,100
21	8	1½	14·14	19	½	1¾	254,500	185,600	656,300	710,400	764,400	602,300	642,800	683,300
	12	1¾	28·86	17½	¾	3	519,500	228,000	924,300	968,600	1,013,000	880,100	913,300	946,500
	8	¾	3·53	20⅜	5/16	1¾	63,600	76,900	393,300	456,500	519,700	330,100	377,500	424,900
22	8	1¾	19·24	19¾	½	1¾	346,400	192,700	769,300	826,800	884,300	711,700	754,900	798,000
	18	1½	31·81	19	¾	3	572,500	246,700	1,028,000	1,080,000	1,132,000	975,700	1,015,000	1,054,000
	8	⅞	4·81	22⅜	5/16	1⅞	86,600	84,300	474,500	550,400	626,300	398,600	455,660	512,500
24	8	1¾	19·24	22	½	1¾	346,400	214,100	848,100	920,000	991,900	776,200	830,100	884,100
	12	2	37·70	20	¾	2½	678,600	311,000	1,218,000	1,275,000	1,332,000	1,161,000	1,204,000	1,247,000
	8	⅞	4·81	24¼	⅜	2¼	86,600	102,500	547,400	637,000	726,600	457,900	525,000	592,200
26	8	2	25·13	23⅝	½	1½	452,400	267,900	1,049,000	1,131,000	1,213,000	966,500	1,028,000	1,090,000
	14	2	43·98	22	¾	2½	791,700	341,000	1,408,000	1,477,000	1,546,000	1,339,000	1,391,000	1,443,000
	8	1	6·28	26¼	⅜	2¼	113,100	110,900	642,300	746,900	851,500	537.700	616,200	694,600
28	12	1¾	28·86	24½	⅝	2½	519,500	261,500	1,136,000	1,225,000	1,313,000	1,047,000	1,114,000	1,180,000
	·16	2	50·27	24	⅞	3	904,800	422,900	1,659,000	1,742,000	1,824,000	1,576,000	1,638,000	1,700,000
	8	1	6·28	28¼	⅜	2¼	113,100	119,200	716,600	837,700	958,800	595,500	686,300	777,200
30	18	1½	31·81	27	⅝	2½	572,500	287,600	1,291,000	1,399,000	1,507,000	1,184,000	1,264,000	1,345,000
	18	2	56·55	26	⅞	3	1,018,000	456,900	1,863,000	1,960,000	2,057,000	1,766,000	1,839,000	1,912,000
	8	1⅛	7·95	30¼	⅜	2	143,100	143,500	840,300	978,800	1,117,000	701,900	805,700	909,600
32	16	1¾	38·48	28½	⅝	2½	692,700	303,200	1,473,000	1,592,000	1,712,000	1,354,000	1,443,000	1,533,000
	20	2	62·83	28	⅞	2¾	1,131,000	535,600	2,116,000	2,229,000	2,341,000	2,004,000	2,088,000	2,173,000
	8	1⅛	7·95	32¼	⅜	2	143,100	152,800	925,200	1,083,000	1,240,000	767,900	885,900	1,004,000
34	24	1½	42·41	31	⅝	2½	763,400	328,400	1,657,000	1,798,000	1,939,000	1,515,000	1,621,000	1,727,000
	21	2	65·97	30	⅞	2¾	1,188,000	572,700	2,280,000	2,409,000	2,539,000	2,150,000	2,247,000	2,344,000
	8	1¼	9·82	34¼	⅜	2	176,700	162,300	1,047,000	1,224,000	1,401,000	870,000	1,003,000	1,136,000
36	20	1¾	48·11	32½	⅝	2½	865,900	344,800	1,829,000	1,984,000	2,138,000	1,675,000	1,790,000	1,906,000
	23	2	72·26	32	⅞	2¾	1,301,000	609,800	2,501,000	2,649,000	2,796,000	2,353,000	2,464,000	2,575,000

TABLE 11. REDUCTION COEFFICIENTS FOR STRESSES IN LONG COLUMNS

Ratio of effective length to least lateral dimension of column	Coefficient*
15	1·0
18	0·9
21	0·8
24	0·7
27	0·6
30	0·5
33	0·4
36	0·3
39	0·2
42	0·1
45	0

* Intermediate values may be obtained by interpolation.

When, in a column having helical reinforcement, the permissible load is based on the core area, the least lateral dimension should be taken as the diameter of the core.

(v) *Determination of ratio of effective length to least lateral dimension of a column.* For the purpose of this clause the effective column length given in Table 12 should be used, where L is the length of the column from floor to floor, or between adequately restrained supports. The effective column length values given in this table are in respect of typical cases only and embody the general principles which should be employed in assessing the appropriate value for any particular column.

The figures given in *Table* 11 may be obtained from the formula

$$C_r = 1{\cdot}5\frac{l}{30d}$$

where C_r = reduction coefficient, l = effective length of column, and d = least lateral dimension of column.

To obtain the effective column length to be used in *Table* 11 it is necessary to assess the degree of end restraint and to determine from *Table* 12 the ratio of the effective length to the length of the column. The length of the column L should be measured between the upper surfaces of two floors affording lateral support to the column. Where other forms of adequate lateral support are used the length of the column may be assumed to be the distance between the centres of the supporting members or the clear distance between supports plus the lateral dimension of the column.

Reinforced concrete inherently affords a greater degree of end fixity in both position and direction than most other structural materials, and it is unlikely that the effective length of any column ordinarily met with in reinforced concrete frame buildings would exceed its actual length.

The effective length with the maximum amount of end restraint is 0·75 of

TABLE 12. EFFECTIVE COLUMN LENGTHS

	Type of column	Effective column length
Columns of one storey.	Properly restrained at both ends in position and direction.	0·75 L
	Properly restrained at both ends in position but not in direction.	L
	Properly restrained at one end in position and direction and imperfectly restrained in both position and direction at the other end.	A value intermediate between L and 2 L depending upon the efficiency of the imperfect restraint.
Columns continuing through two or more storeys.	Properly restrained at both ends in position and direction.	0·75 L
	Properly restrained at both ends in position and imperfectly restrained in direction at one or both ends.	A value intermediate between 0·75 L and 1·00 L depending upon the efficiency of the directional restraint.
	Properly restrained at one end in position and direction and imperfectly restrained in both position and direction at the other end.	A value intermediate between L and 2 L depending upon the efficiency of the imperfect restraint.

the real length. Consequently for all ordinary cases the maximum permissible length of a column before a reduction of load is necessary on account of buckling will vary between 15 and 20 times the least lateral dimension. Columns as ordinarily employed in buildings, including even external corner columns in the top floor, should in all but exceptional cases come within these limits.

It should be noted that in long columns the permissible bending stresses are [Clause 312 (*c*)] equal to the values given in Clause 303 multiplied by the appropriate reduction coefficient from *Table* 11. Since the amplification of stress due to incipient buckling of the column affects most the stresses at mid-length of the column, the reduction coefficient need not be considered in determining the permissible stress for sections within one-eighth of the length of the column from the centre-line of the beams.

(*b*) BENDING MOMENTS IN COLUMNS.

Bending moments in internal columns supporting an approximately symmetrical arrangement of beams and loading need not be calculated except in the case of flat slab construction. See clauses 313 to 323.

Bending moments in external columns and in internal columns supporting an arrangement of beams and loading not approximately symmetrical should be calculated and provided for.

The expressions given in Table 13 below may be used for estimating the moments :—

TABLE 13. MOMENTS IN COLUMNS

	Moments for frames of one bay	Moments for frames of two or more bays
External (and similarly loaded) columns Moment at foot of upper column	$M_e\frac{K_u}{K_l + K_u + 0{\cdot}5\ K_b}$	$M_e\frac{K_u}{K_l + K_u + K_b}$
Moment at head of lower column	$M_e\frac{K^l}{K_l + K_u + 0{\cdot}5\ K_b}$	$M_e\frac{K_l}{K_l + K_u + K_b}$
Internal columns Moment at foot of upper column		$M_{es}\frac{K_u}{K_l + K_u + K_{b1} + K_{b2}}$
Moment at head of lower column		$M_{es}\frac{K_l}{K_l + K_u + K_{b1} + K_{b2}}$

where M_e is the bending moment at the end of the beam framing into the column, assuming fixity at the connection ;
M_{es} is the maximum difference between the moments at the ends of the two beams framing into opposite sides of the column, each calculated on the assumption that the ends of the beams are fixed and assuming one of the beams unloaded ;
K_u is the stiffness of the upper column ;
K_l is the stiffness of the lower column ;
K_b is the stiffness of the beam ;
K_{b1} is the stiffness of the beam on one side of the column ;
K_{b2} is the stiffness of the beam on the other side of the column.

For the purposes of this table, the 'stiffness' of a member may be obtained by dividing the moment of inertia of a cross-section by the length of the member, provided that the member is of constant cross-section throughout its length.

The equations for the moment at the head of the lower column may be used for columns in a topmost storey by taking K_u as zero.

Where the bending moment is calculated in the internal columns it is permissible to take into account the reduction in load resulting from the beam on one side of the column being fully loaded and the beam on the other side being loaded with dead load only.

(*c*) COMBINED AXIAL AND BENDING STRESS.

The stresses in columns due to the combination of direct load and bending action should not exceed the permissible stresses for bending, multiplied by the appropriate reduction coefficient from Table 11.

Where, due to bending, the maximum stresses in the column occur at the ends of the column, the permissible stress may be determined without reference to the reduction coefficient for sections within one-eighth of the column length L from the centre line of the beams.

Bending in Columns.—The formulæ given for the estimation of bending moments in external columns are based on the following assumptions:

(*a*) That the points of intersection of the members remain fixed in position relative to one another, in other words changes in length of the members are ignored.

(*b*) That in the case of frames of two or more bays the ends of the columns and beams remote from the intersection under consideration are fixed in position and direction (see diagrams in *Table* 30A).

(*c*) That in the case of frames of one bay the ends of the columns remote from the beam under consideration are fixed in position and direction (see *Table* 30A), and

(*d*) That in the case of frames of one bay the columns at each end of the span are identical.

Assumptions (*a*) and (*b*) apply also to the formulæ given for the moments in internal columns with unbalanced beams.

Although the formulæ in *Table* 13 are not strictly accurate, they are sufficiently approximate for most practical cases. The values obtained will be too low if the remote end of the beam is not rigidly fixed or when bending moments of similar sign (either clockwise or anti-clockwise) are applied to the columns by beams above and below. The values will be too high when the columns are not rigidly fixed at their remote ends.

Where beams frame into columns from two directions at right-angles it is clear also that the torsional resistance of the beams should be taken into account in any attempt at accurate calculation.

In *Table* 30A the proportion of the fixed-end bending moment resisted by the beam at the junction with an external column is given for a range of column-to-beam stiffnesses. For the case of uniform loading the moment coefficients (S), where the bending moment on the beam at the joint is equal to $-\frac{Wl^2}{S}$, are also given. Having calculated the bending moment on the beam at the end column junction, bending moments at other supports of continuous beams of equal spans can be found by the use of *Fig.* 9A.

CALCULATION OF STIFFNESS.—The stiffness is defined as the value obtained by dividing the moment of inertia of a member by its length. It may be regarded as proportional to the moment necessary to rotate one end of a member through unit angle when the member is supported at both ends, the remote end being fixed in direction.

It will be noted from Clause 306 (*c*) that the moments of inertia, and therefore the stiffnesses of members, may be calculated by one of three methods, provided that the same method is used throughout the calculation. The methods are:

(*a*) Using the whole area of the section, ignoring the reinforcement,

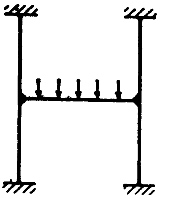
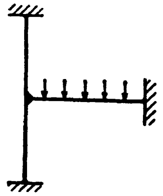
(*b*) As (*a*) but including the reinforcement at its appropriate modular ratio, and

(*c*) Using the compression area of the concrete, ignoring the tension area, and allowing for the reinforcement at its appropriate modular ratio.

These will generally lead to different results, although in normal cases the difference is not likely to affect design considerably.

Fig. 13A gives values of a coefficient C which enable moments of inertia

TABLE 30A.

STIFFNESS RATIO				
$\left(\frac{K_u + K_l}{K_b}\right)$	$\frac{K_l + K_u}{K_l + K_u + \frac{K_b}{2}}$	MOMENT FACTOR S	$\frac{K_l + K_u}{K_l + K_u + K_b}$	MOMENT FACTOR S
0·2	0·286	42	0·167	72
0·3	0·375	32	0·231	52
0·4	0·444	27	0·286	42
0·6	0·545	22	0·375	32
0·8	0·615	19·5	0·444	27
1·0	0·667	18	0·500	24
2·0	0·800	15	0·667	18
3·0	0·857	14	0·750	16
4·0	0·889	13·5	0·800	15
6·0	0·923	13	0·857	14
10·0	0·952	12·6	0·909	13·2

Negative bending moment in beam at junction $= \frac{wl^2}{S}$.

to be obtained for tee-beams when the reinforcement is ignored. As an indication of the manner in which inclusion of the reinforcement affects the value of the moments of inertia, the following figures may be of interest. These figures give the ratio of the moments of inertia of rectangular reinforced concrete sections to similar plain sections for various percentages of reinforcement, and are based on the assumption that symmetrical reinforcement is placed at a distance 0·4d from the centroid of the section and that the modular ratio is 15.

EFFECT OF VARIOUS PERCENTAGES OF SYMMETRICAL REINFORCEMENT ON MOMENT OF INERTIA OF RECTANGULAR SECTIONS.

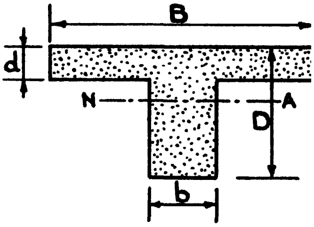
Percentage of longitudinal reinforcement	Ratio of moments of inertia (reinforced to plain)
0	1·0
1	1·3
2	1·5
3	1·8
4	2·1
6	2·6
8	3·1

MOMENTS OF INERTIA

ON BEAM SECTIONS (PLAIN CONCRETE)

$I_{NA} = CbD^3$.

VERTICAL ORDINATES GIVE
VALUES OF COEFFICIENT C.



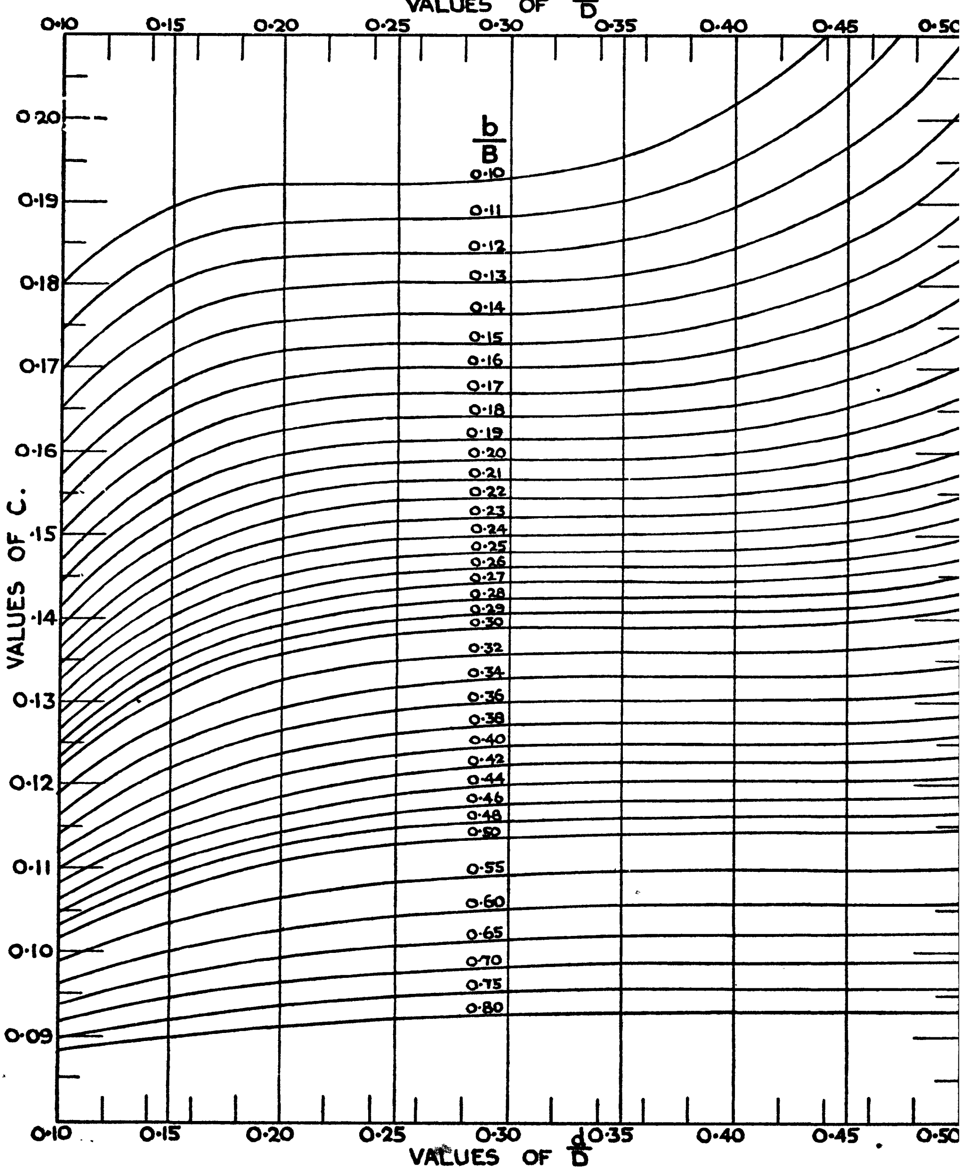


FIG. 13A.

A similar effect will be found in the case of beams, method (*b*) in all cases giving higher values of the moment of inertia than either (*a*) or (*c*), and (*c*) usually lying between (*a*) and (*b*).

For the length of the member it is in general sufficiently accurate to use the distance between the intersections of its axis with the axes of the members into which it frames.

For members of varying section special treatment is necessary, since the stiffness cannot be defined in simple general terms. A full analysis of such cases is not given, but methods of dealing with them may be found in text books. In such cases the fixed-end bending moments as well as the stiffnesses will need special consideration. For example, for a haunched beam of rectangular section the depth of which at the centre is one-half that at the supports and with the haunch extending for 0·2 of the span, the fixed-end bending moment under uniform load will be approximately $\frac{Wl}{10}$ instead of $\frac{Wl}{12}$ for a uniform section, and the stiffness will be approximately doubled. This will decrease the ratio of the sum of the stiffnesses of the columns to the stiffness of the beam, and will therefore reduce the sum of the bending moments to be resisted by the columns in most cases to values much less than those obtained if the beam is without haunches. Haunches on T-beams can safely be ignored in most practical cases.

It will be seen that the sum of the bending moments on the columns—that is equal to the bending moment on the beam—approaches a limiting value equal to the fixed-end bending moment for the beam as the ratio of the sum of the stiffnesses of the columns to the stiffness of the beam approaches infinity. This is shown in *Table* 30A.

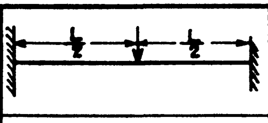
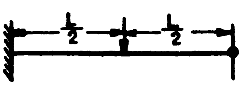
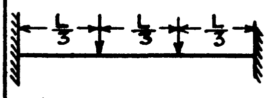
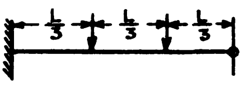
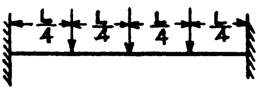
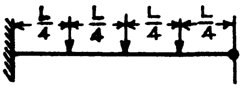
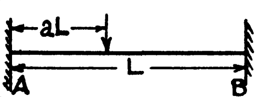
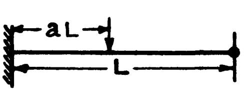
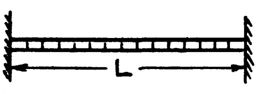
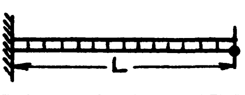
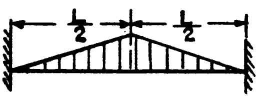
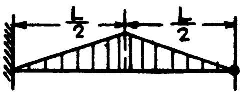
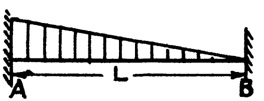
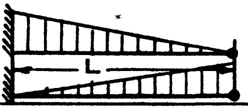
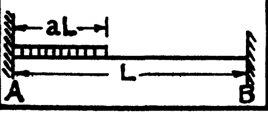
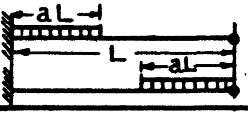
In the case of stiff columns and relatively flexible beams, which may occur in the lower stories of buildings, the bending moment on the beam may often approach closely to the fixed-end bending moment. In such cases, however, the margin between the permissible direct and permissible bending stresses is usually sufficient to provide for this bending moment on the columns, that is about one-half the fixed-end bending moment for the beam on each column.

In the cases of columns in intermediate and top stories, however, the external columns will frequently need special provision to resist bending. Such provision may be made by increasing (*a*) the breadth or width of the column, or (*b*) the quantity of reinforcement, or (*c*) the proportion of cement, or by any combination of these three methods. When method (*a*) is used the bending moment taken by the column is increased in all cases and it is necessary to ascertain whether the section is capable of resisting this increased bending moment. Using method (*b*), the bending moment to be resisted by the column is increased only when the calculation of the moment of inertia of the section allows for the reinforcement. When method (*c*) is used the bending moment to be resisted by the column needs no further attention.

In making calculations of the bending moments on columns the values of the fixed-end bending moments given in *Table* 31A may be employed.

It should be noted that the permissible compressive stress in the steel in columns subjected to bending is the compressive stress in the concrete multiplied by the modular ratio, whereas in axially-loaded columns the permissible stress in the steel is fixed according to the grade of steel employed [see Clause 303 (*b*)], The resistance of columns subjected to bending must, therefore, be calculated by using the modular ratio. The extreme fibre stress in the concrete may be taken up to the permissible value in bending as given in Clause 303 (*a*), provided that the load does not exceed the value permitted by Clause 312 (*a*). When subjected to bending, helically-reinforced concrete columns must be treated as are columns with lateral ties, that is by ignoring the effect of the helix and calculating the stresses on the entire cross section.

TABLE 31A.—FIXED-END MOMENTS (M_e).

	BOTH ENDS FIXED		ONE END FIXED **	
	TYPE OF LOADING	* COEFFICIENT OF WL	TYPE OF LOADING.	COEFFICIENT OF WL
POINT LOADS.		$-\frac{1}{8}$		$-\frac{3}{16}$
		$-\frac{1}{9}$		$-\frac{1}{6}$
		$-\frac{5}{48}$		$-\frac{5}{32}$
		at A $-a(1-a)^2$ at B $-a^2(1-a)$		$-a(1-a)(1-\frac{a}{2})$
DISTRIBUTED LOADS.		$-\frac{1}{12}$		$-\frac{1}{8}$
		$-\frac{5}{48}$		$-\frac{5}{32}$
		at A $-\frac{1}{10}$ at B $-\frac{1}{15}$		$-\frac{2}{15}$ $-\frac{7}{60}$
		at A $-\frac{a(6-8a+3a^2)}{12}$ at B $-\frac{a^2(4-3a)}{12}$		$-\frac{a(4-4a+a^2)}{8}$ $-\frac{a(2-a^2)}{8}$

* W = TOTAL LOAD ON SPAN.

** WITH ONE END FIXED STIFFNESS SHOULD BE TAKEN AS 3/4 I/L.

The following example illustrates the method of obtaining bending moments at the junctions of columns and beams in *Fig.* 14A by the use of *Table* 30A and *Fig.* 13A.

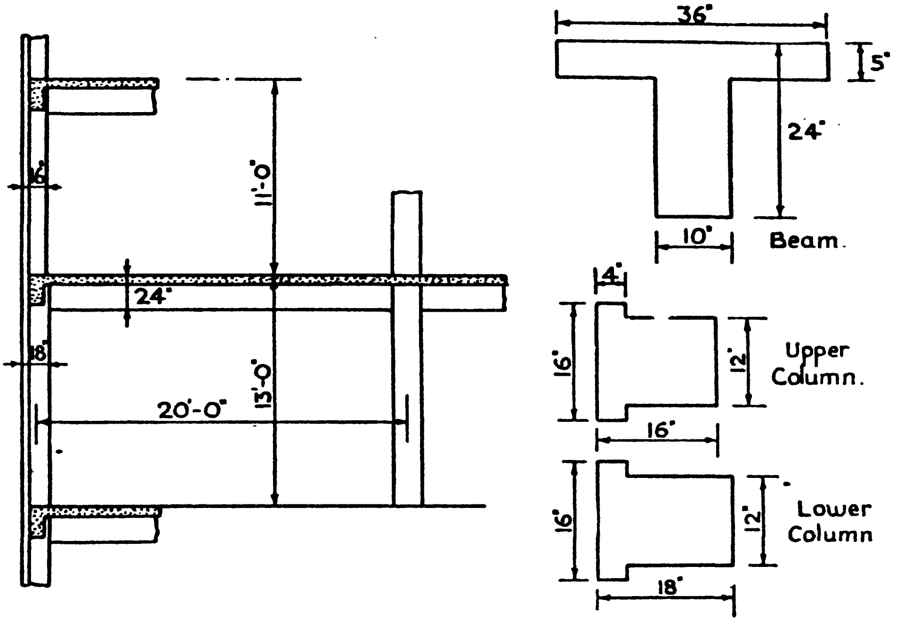


FIG. 14A.—JUNCTIONS OF BEAMS WITH COLUMNS.

Upper Column :

$\frac{b}{B}=\frac{12''}{16''}=0{\cdot}75 \qquad \frac{d}{D}=\frac{4''}{16''}=0{\cdot}25 \qquad I \text{ (from } Fig. \text{ 13A)} =0{\cdot}095bD^3=4665 \text{ in.}^4$

$\text{Stiffness } K_u=\frac{4665}{11 \text{ ft.} \times 12}=35{\cdot}4.$

Lower Column :

$\frac{b}{B}=\frac{12''}{16''}=0{\cdot}75 \qquad \frac{d}{D}=\frac{4''}{18''}=0{\cdot}222. \qquad I \text{ (from } Fig. \text{ 13A)} =0{\cdot}0945bD^3=6610 \text{ in.}^4$

$\text{Stiffness } K_l=\frac{6610}{13 \text{ ft.} \times 12}=42{\cdot}4.$

Beam :

$\frac{b}{B}=\frac{10''}{36''}=0{\cdot}278 \qquad \frac{d}{D}=\frac{5''}{24''}=0{\cdot}208 \qquad I \text{ (from } Fig. \text{ 13A)} =0{\cdot}1405bD^3=19{,}400 \text{ in.}^4$

$\text{Stiffness } K_b=\frac{19400}{20 \text{ ft.} \times 12}=80{\cdot}9.$

To obtain moment factor :

$$\frac{K_l+K_u}{K_l+K_u+K_b}=\frac{42{\cdot}4+35{\cdot}4}{42{\cdot}4+35{\cdot}4+80{\cdot}9}=0{\cdot}49.$$

Interpolating values in *Table* **30A** :

$S=24{\cdot}5$

Negative bending moment in beam at junction, $\frac{wl^2}{24{\cdot}5}$

B.M. at foot of upper column, $M_e\left(\frac{K_u}{K_l+K_u+K_b}\right)=0{\cdot}223M_e=0{\cdot}223\times\frac{wl^2}{12}=\frac{wl^2}{54}$.

B.M. at head of lower column, $M_e\left(\frac{K_l}{K_l+K_u+K_b}\right)=0{\cdot}267M_e=0{\cdot}267\times\frac{wl^2}{12}=\frac{wl^2}{45}$.

SOLID REINFORCED CONCRETE FLAT SLAB CONSTRUCTION

313. General. The following clauses, 314 to 323 inclusive, apply only when the conditions (*a*) and (*b*) below are satisfied.

(*a*) *Limitations regarding numbers and shape of a series of panels.* This applies only to floors and roofs comprising a series of rectangular panels of approximately constant thickness arranged in at least three rows in each direction, the ratio borne by the length of a panel to its width not exceeding 4 : 3.

The lengths and/or widths of any two adjacent panels in a series should not differ by more than 10 per cent of the greater length or width.

End spans may be shorter, but not longer, than interior spans. Where adjacent spans differ, the length should always be taken as that corresponding to the longer span, in calculating the bending moments.

(*b*) *Limitations regarding drop.* The drops should be rectangular in plan, and have a length in each direction not less than one-third of the panel length in that direction.

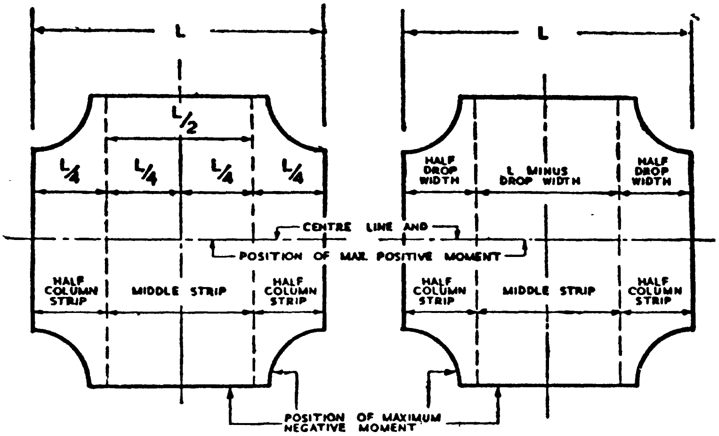
For exterior panels the width of drop at right angles to the non-continuous edge measured from the centre line of the columns should be equal to one-half the width of drop for interior panels.

The design of flat slabs is based almost wholly upon practical considerations and experience, initially in the United States of America and more recently in this country. Owing to the absence of an easily-applied basic theory it is necessary, for general use, to impose greater restrictions in flat-slab design than for ordinary beam and slab construction, as a result of which the selection of sections and the arrangement of reinforcement are kept within narrow limits. It should not be assumed, however, that it is possible to design flat slabs only within these limits. Provided that proper allowances and adjustments are made to the bending moment and other relevant factors, it is possible to design satisfactory flat slabs which fall outside the scope of this part of the Code in a great many respects. Such structures are not excluded by the Code, which gives recommendations which apply for certain restricted conditions but does not specify that all flat slabs must satisfy these conditions.

It should be noted that the rules cover flat slabs with or without drops. Where drops are used, no upper limit is stated for the length of drop, but it appears that the recommendations in Clause 316 are not intended to apply for a length of drop greater than one-half the corresponding length of panel.

314. Division of panels. (See Fig. 5.) (*a*) *Column strip.* The width of the column strip should be taken as one-half the width of the panel, excepting that where drops are used, it may be taken as the width of the drop.

(*b*) *Middle Strip.* The width of the middle strip should be taken as one-half the width of the panel, excepting that where drops are used, and the column strip is taken as the width of the drop, the width of the middle strip should be taken as the difference between the panel width and the drop width.



(*a*) Where drops are not used. (*b*) Where drops are used.

FIG. 5.—DIVISION OF FLAT SLAB PANELS INTO COLUMNS AND MIDDLE STRIPS.

Considering a number of square panels it will be seen that the width of the column strips, where a drop is employed, can vary between $\frac{L}{3}$ and $\frac{L}{2}$, and the middle strip can vary from $\frac{L}{2}$ to $\frac{2L}{3}$, each strip having a permissible variation in width of $\frac{L}{6}$. *Fig.* 15A (page 92) shows the limits of variation for the strips, drops, and column heads (the areas of variation are shaded).

315. Notation for flat slab construction. In the following clauses and formulæ relating to flat slabs, L_1 is the length of the panel in the direction of span, L_2 is the width of the panel at right angles measured in each case from centre line to centre line of columns, L is the average of L_1 and L_2, D is the diameter of the column head and *w* is the total load per unit area. The formulæ give moments on the whole width of the strip.

316. Moments in interior panels fully continuous. (*a*) *Moments to be used in design.* For interior panels, fully continuous, the bending moments to be provided for at the positions indicated in Fig. 5, i.e.

(i) Positive moment along the centre lines of the panel;

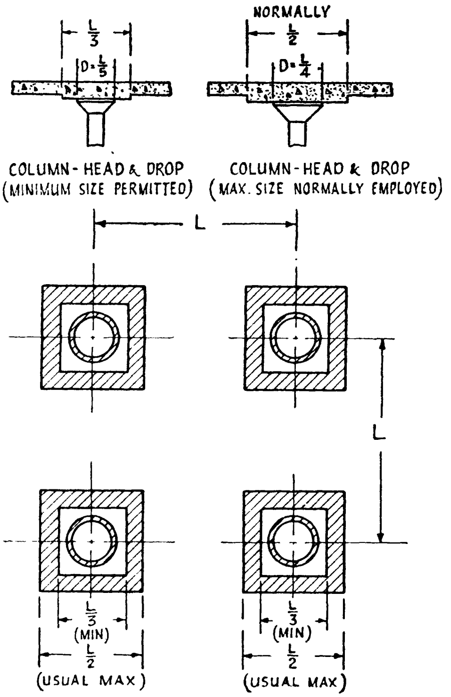
(ii) Negative moment along the edges of the panel on lines joining the centres of the columns and around the perimeter of the column heads, should be:—

FOR PANELS WITHOUT DROPS:

	Positive moment	*Negative moment*
Column strip	$0{\cdot}022wL_2\left(L_1 - \frac{2D}{3}\right)^2$ . . .	$0{\cdot}042wL_2\left(L_1 - \frac{2D}{3}\right)^2$
Middle strip	$0{\cdot}018wL_2\left(L_1 - \frac{2D}{3}\right)^2$ . . .	$0{\cdot}018wL_2\left(L_1 - \frac{2D}{3}\right)^2$

FLAT SLABS.

Column-head and Drops.


NORMALLY
$\frac{L}{3}$
$D=\frac{L}{5}$
$\frac{L}{2}$
$D=\frac{L}{4}$
COLUMN-HEAD & DROP (MINIMUM SIZE PERMITTED)
COLUMN-HEAD & DROP (MAX. SIZE NORMALLY EMPLOYED)
L
L
$\frac{L}{3}$ (MIN)
$\frac{L}{2}$ (USUAL MAX)
$\frac{L}{3}$ (MIN)
$\frac{L}{2}$ (USUAL MAX)

(Shaded portions show variations in sizes.)

FIG. 15A.

FOR PANELS WITH DROPS :

	Positive moment		*Negative moment*
Column strip	$0{\cdot}022wL_2\left(L_1 - \frac{2D}{3}\right)^2$	. . .	$0{\cdot}046wL_2\left(L_1 - \frac{2D}{3}\right)^2$
Middle strip	$0{\cdot}016wL_2\left(L_1 - \frac{2D}{3}\right)^2$	. . .	$0{\cdot}016wL_2\left(L_1 - \frac{2D}{3}\right)^2$

(*b*) *Special case of middle strip wider than half width of panel.* Where the column strip is taken as equal to the width of the drop, and the middle strip is thereby increased in width to a value greater than half the width of the panel, the moments to be taken on the middle strip should be increased in proportion to its increased width. The moments to be taken by the column strip may then be decreased by an amount such that there is no reduction in either the total positive or the total negative moments taken by the column strip and middle strip.

For square panels, $L_1 = L_2 = L$. Also, from Clause 323, the diameter of internal column heads is restricted to between $\frac{L}{5}$ and $\frac{L}{4}$. The positive and negative bending moments may therefore be expressed in terms of the total unit load and the span. In square panels without drops, the widths of the strips on which these moments occur are proportional to the span, and it is therefore possible to express the bending moments in the limiting cases as for ordinary slabs (average bending moments per unit width of slab). For comparative purposes these bending moments are given in *Table* 32A.

TABLE 32A.—BENDING MOMENTS IN INTERMEDIATE PANELS FULLY CONTINUOUS WITHOUT DROPS.

		Positive bending moments per unit width	Negative bending moments per unit width
COLUMN STRIP	Smallest head	$+\frac{wL^2}{30\cdot3}$	$-\frac{wL^2}{15\cdot9}$
	Largest head.	$+\frac{wL^2}{32\cdot7}$	$-\frac{wL^2}{17\cdot1}$
MIDDLE STRIP	Smallest head	$+\frac{wL^2}{37\cdot0}$	$-\frac{wL^2}{37\cdot0}$
	Largest head.	$+\frac{wL^2}{40\cdot0}$	$-\frac{wL^2}{40\cdot0}$

Where a drop of width less than $\frac{L}{2}$ is used and the width of the column strip is assumed to be equal to the width of the drop, the bending moments given in Clause 316 (*a*) require adjustment in accordance with Clause 316 (*b*). The bending moments per unit width in the column strip also depend on the width assumed. The variations are shown in *Table* 33A.

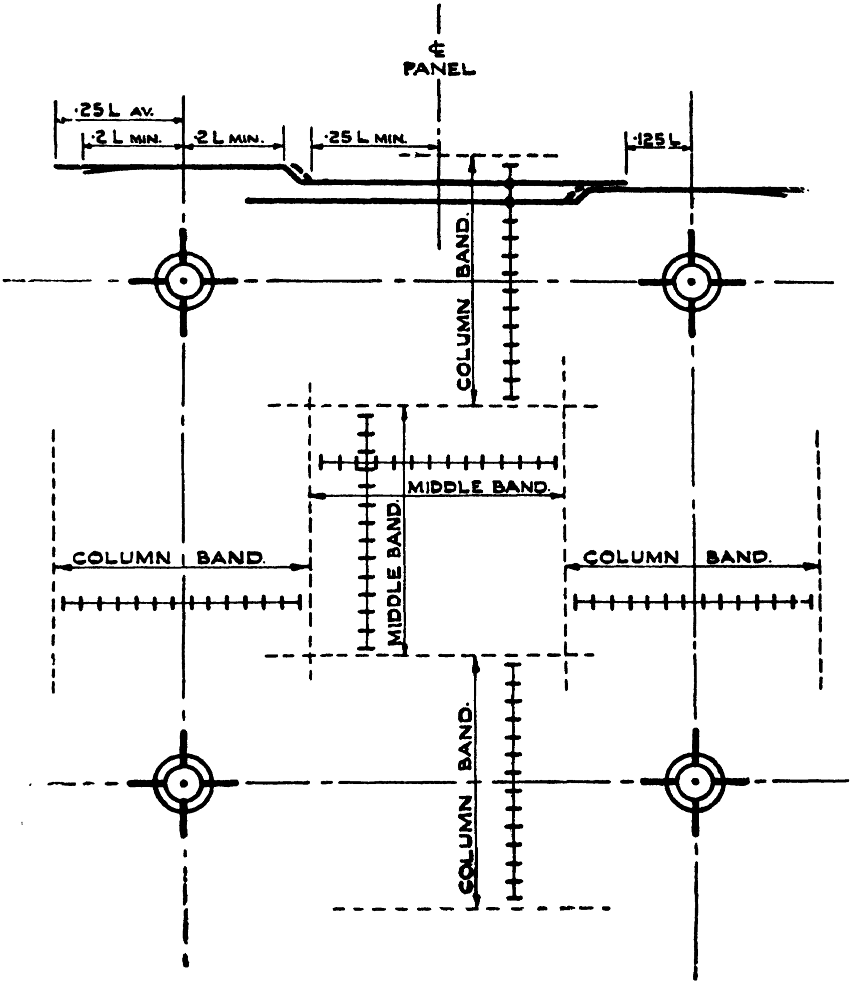
TABLE 33A.—BENDING MOMENTS IN INTERMEDIATE PANELS FULLY CONTINUOUS WITH DROPS.

	Positive bending moments per unit width	Negative bending moments per unit width
COLUMN STRIP.		
Smallest head and narrowest column strip .	$+\frac{wL^2}{26\cdot6}$	$-\frac{wL^2}{10\cdot9}$
Largest head and widest column strip . .	$+\frac{wL^2}{32\cdot7}$	$-\frac{wL^2}{15\cdot5}$
MIDDLE STRIP.		
Smallest head	$+\frac{wL^2}{41\cdot6}$	$-\frac{wL^2}{41\cdot6}$
Largest head	$+\frac{wL^2}{45\cdot0}$	$-\frac{wL^2}{45\cdot0}$

For comparison with these bending moments in flat slabs it may be noted that for a square panel continuous on all four sides over the supporting beams the positive bending moment at mid-span is $\frac{wL^2}{40}$ and the negative bending moment over the supports is $\frac{wL^2}{30}$ (see *Table* 10). Note that the bending moments on strips of unit width tabulated cannot be employed generally for the purpose of ascertaining the reinforcement required, since the factors given are for the limiting cases only. Also, in the case of four-way reinforcement the width of the direct bands, required by Clause 317 (*b*), will not usually be the same as the assumed width of the strip upon which the unit bending moments are based. Further, it will be seen that while a decrease in the width of column strip results always in an increase of bending moment per unit width for this strip, there is no variation in the bending moments per unit width for the middle strip. In this connection it should be realised that the same total amount of bending moment is to be resisted across both the middle and column strips, the only modification being in the distribution of bending moment across the column strip.

> **317. Widths of reinforcing bands.** (*a*) *Slabs reinforced in two directions.* In slabs reinforced in two directions only, the reinforcement in each strip should be so disposed that the strip is reinforced over its full width.
>
> (*b*) *Slabs reinforced in four directions.* In slabs reinforced in four directions, the width of each direct band of reinforcement should be two-fifths of the panel width at right angles to the direction of the reinforcement, and the width of each diagonal band should be one-half the panel length—or one-half the average panel length, in the case of panels which are not square.
>
> (*c*) *Apportionment of reinforcement in four-way systems.* In four-way systems the reinforcement should be apportioned as follows :
>
> (i) the reinforcement in the direct band should take the entire positive moment in the column strip ;
>
> (ii) the reinforcement in the diagonal bands should take the entire positive moment in the middle strip ;
>
> (iii) the reinforcement in the direct band, plus the reinforcement in the diagonal bands (the effective area of which should be calculated as described in clause 102) should take the negative moment in the column strip ;
>
> (iv) additional reinforcement should be provided to take the negative moment in the middle strip.

The disposition of the various bands of reinforcement as defined in Clause 317 (*a*) and (*b*) is shown in *Figs.* 16A and 17A. With the two-way system the necessary top reinforcement transverse to the central portions of the column strips is provided by the bent-up bars from the middle strips. In the case of the four-way system where the positive bending moment on the middle strips is resisted by diagonal reinforcement, it is necessary to provide additional reinforcement in the form of " clamps " to resist the negative bending moments on the middle strip.



Arrangement of Bars and Bends in Column and Middle Bands to be Similar.
FIG. 16A.—TWO-WAY REINFORCEMENT.

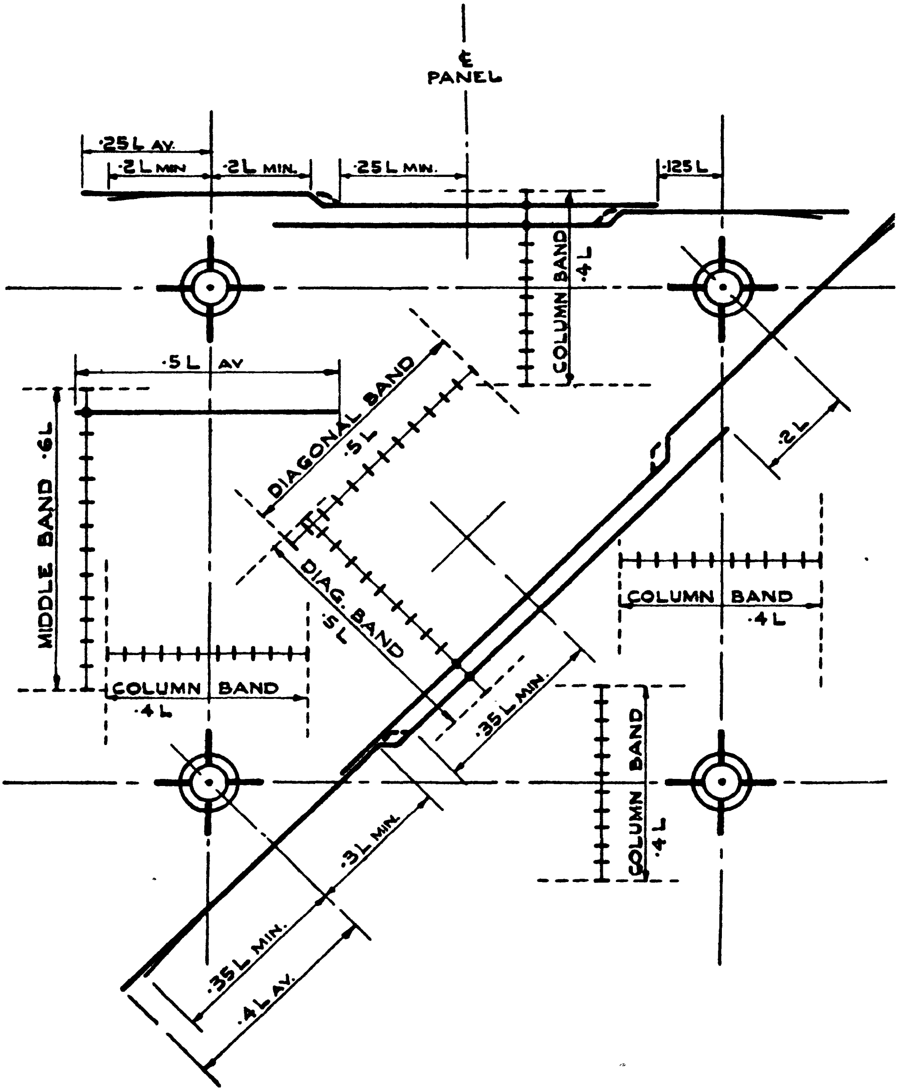


FIG. 17A.—FOUR-WAY REINFORCEMENT.

318. Thickness of slab. The total thickness of the slab should in no case be less than the greatest of the following values :—

(*a*) 5 in.

(*b*) L/32 for end panels without drops.

(*c*) L/36 for interior panels, fully continuous, without drops and for end panels with drops.

(*d*) L/40 for interior panels, fully continuous, with drops.

With large panels the dead weight may form a considerable proportion of the total load, and in such cases the determination of the thickness of the slab becomes an important factor. The minimum thicknesses specified in Clause 318 will often so control the design that, except for very heavy superimposed load, there will be little advantage in the use of large percentages of reinforcement or for concrete of very high strength. As an example, the case of a slab reinforced with 1 per cent. of steel may be considered. For any of the specified concretes not weaker than 1 : 1½ : 3 of the permitted lower grade, more than 1 per cent. of reinforcement is required if the maximum permissible compressive stress is to be realised and if the stress in the reinforcement is not to exceed 18,000 lb. per square inch. The permissible bending moment per foot width of slab for this condition, that is 1 per cent. of reinforcement and $t = 18{,}000$ lb. per square inch, is $155 \times 12d^2$ lb.-in. From considerations of resistance to the bending moment alone, therefore, the effective depth of the slab is obtained by equating $155 \times 12d^2$ to the bending moments required by Clause 316. Values for the limiting cases computed in this way are given in *Table* 34A.

Table 34A.—Flat Slabs—Effective Depths.

Case and condition	Effective Depth (in.). L in ft. and w in lb. per square foot	Bending moment determining effective depth
(1) Slab without Drops:		
Smallest column head	0·0202 $L\sqrt{w}$	Negative moment in column strip
Largest column head	0·0194 $L\sqrt{w}$	
(2) Slab with Square or Rectangular Drops:		
Drop:		
Smallest column head and narrowest column strip	0·0244 $L\sqrt{w}$	
Largest column head and widest column strip	0·0204 $L\sqrt{w}$	
Slab:		
Smallest column head and narrowest column strip	0·0156 $L\sqrt{w}$	Positive moment in column strip
Largest column head and widest column strip	0·0141 $L\sqrt{w}$	

Note.—Tensile reinforcement will normally consist of layers of bars at right angles. The effective depth (d) will be measured to the innermost layer or layers, and in determining the minimum thickness of the slab or drop allowance must be made for the outer layer of bars (parallel to the cross-section considered) as well as the cover of concrete.

For interior panels without drops, the minimum total thickness $\frac{L}{36}$ specified in Clause 318 (*c*) will usually be greater than that resulting from the effective depths given in *Table* 34A for superimposed loads up to about 100 lb. per square foot. Where drops are provided, the minimum thickness given in Clause 318 (*d*) will normally govern the thickness, except for superimposed loads well in excess of 100 lb. per square foot.

The Code does not specify limits for the thickness of the drop. As a general rule, however, the thickness of the drop, measured from the upper surface of the slab, should not be less than 25 per cent. or more than 50 per cent. greater than the thickness of the slab.

319. Arrangement of reinforcement.

(*a*) *Slabs reinforced in two directions.*

(i) In each strip or band 40 per cent of the positive reinforcement should be continuous in the lower part of the slab, and extend to within a distance of 0·125 L, measured from the line joining the column centres.

(ii) The negative reinforcement should extend in the top of the slab into adjacent panels for an average distance, measured from the line joining the column centres, of not less than 0·25 L, and no bar less than 0·2 L.

(iii) The full area of negative reinforcement should be provided for a distance measured from the line joining the column-centres not less than 0·2 L. The full area of positive reinforcement should be provided for a distance, measured from the centre line of the panel, of not less than 0·25 L.

(*b*) *Slabs reinforced in four directions.*

(i) For direct bands, the rules above (for slabs reinforced in two directions) should be applied.

(ii) In each diagonal band, 40 per cent of the positive reinforcement should be continuous in the lower part of the slab, and should extend to within a distance of 0·2 L, measured from a line through the column centre at right angles to the direction of the band.

(iii) The negative reinforcement in diagonal bands should extend in the top of the slab into adjacent panels for an average distance of 0·4 L beyond a line through the column centre at right angles to the direction of the band, and no bar less than 0·35 L.

(iv) In each diagonal band the full area of negative reinforcement should be provided for a distance not less than 0·3 L, measured from a line through the column centre at right angles to the direction of the band. The full area of positive reinforcement should be provided for a distance not less than 0·35 L, measured from a line through the centre of the panel at right angles to the direction of the band.

(v) The additional reinforcement required to take the negative moment in the middle strip should extend for a distance not less than 0·25 L on each side of the line joining the column centres.

Since only small percentages of tensile reinforcement are necessary for flat slabs in view of the minimum thicknesses specified in Clause 318, compression bars are not usual and the resistance to negative bending moments is obtained by bending up some of the bars provided for the positive bending moments, and the addition of bars over the areas specified in Clause 319. The position of bends and stopping-off bars to comply with this Clause is illustrated in *Fig.* 16A for two-way reinforcement and in *Fig.* 17A for four-way reinforcement.

Although Clauses 319 (*a*) (i) and 319 (*b*) (ii) do not require more than 40 per cent. of the reinforcement in the bottom of the slab to be continuous, it will usually be found more convenient to extend one-half of the bars to within the prescribed distances, and to bend up the other half.

Where, in two-way systems, straight bars are needed in addition to the bent-up bars to provide the reinforcement necessary to resist negative bending moments in the column strips, they should be in the form of " clamps " having an average length of 0·5 L and projecting at least 0·2 L on either side of a line joining the column centres.

In four-way systems, similar clamps to resist the negative bending moment in the middle strip [Clause 318 (*c*) (iv)] should extend at least 0·25 L on each side of the line joining the column centres.

With the bars extending to the positions shown in *Fig.* 16A and 17A, the ends may be left plain [except for end panels, see Clause 320 (*c*)]. The ends of the bars and the position of the bends should be staggered to avoid the occurrence of continuous cracks.

320. Moments on side or end panels not fully continuous. For side or end panels in which the slab is not continuous upon one edge or upon two adjacent edges :—

(*a*) The positive moments to be used for sections parallel with the discontinuous edges (reinforcement at right angles to the edges) should be greater by 25 per cent than those given in clause 316.

(*b*) At the discontinuous edges, the negative moment (reinforcement at right angles to the edges) in the column strip should be taken as not less than 90 per cent, and in the middle strip as not less than 60 per cent of those given in clause 316 (*a*).

(*c*) At all discontinuous edges, the positive and negative reinforcement should extend to within 3 in. of the edge of the panel, and should be provided with U hooks.

(*d*) Where end spans are shorter than interior spans, the moments given in paragraphs (*a*) and (*b*) of this clause may be suitably modified.

The increase of 25 per cent. to the positive bending moment in the span of corner and intermediate end panels, required by Clause 320 (*a*), is required for the same reason that the bending moments in the end spans of continuous beams and slabs are increased. It will be apparent that the spans of the end panels can be made smaller if desired so that the increased bending moment calculated on the smaller span is equal to that in the panel adjacent, thus allowing the use of the same thickness of slab and the same amount of reinforcement.

321. Openings in panels. Except for openings complying with paragraphs (*a*), (*b*) and (*c*) of this clause, openings should be completely framed on all sides with beams to carry the loads to the columns, and an opening should not encroach upon a column head or drop.

(*a*) Openings of a size such that the greatest dimension in a direction parallel with a central line of the panel does not exceed 0·4 L may be formed in the area common to two intersecting middle strips, provided the total positive and negative moments be maintained as specified in clause 316 (*a*) and provided these total positive and negative moments be redistributed between the remaining principal design sections to meet the changed conditions.

(*b*) Openings of aggregate length or width not exceeding one-tenth of the width of the column strip may be made in the area common to two column strips, provided that the reduced sections are capable of carrying the appropriate moments specified in clause 316 (*a*).

(*c*) Openings of aggregate length or width not exceeding one-quarter of the width of the strip may be made in any area common to one column strip and one middle strip, provided that the reduced sections are capable of carrying the appropriate moments specified in clause 316 (*a*).

It should be noted that in some cases the redistribution of bending moment adjacent to openings in a panel may lead to the necessity for compressive reinforcement.

322. Shearing stresses. The shearing stress in the slab or drop, upon a vertical section at a distance equal to the effective depth from the column head, and the shearing stress upon a vertical section along the perimeter of the drop (where used) should not exceed that permissible.

In calculating the resistance to shearing around a column head, the load causing the shearing force should be reckoned as the total dead and live load on the panel, excluding the area supported by the column-head. Suitable adjustment should be made where panels are unequal in dimensions.

Similarly for the resistance to shearing along the perimeter of the drop, the load causing the shearing force should be reckoned as the total dead and live load on the panel less that on the drop.

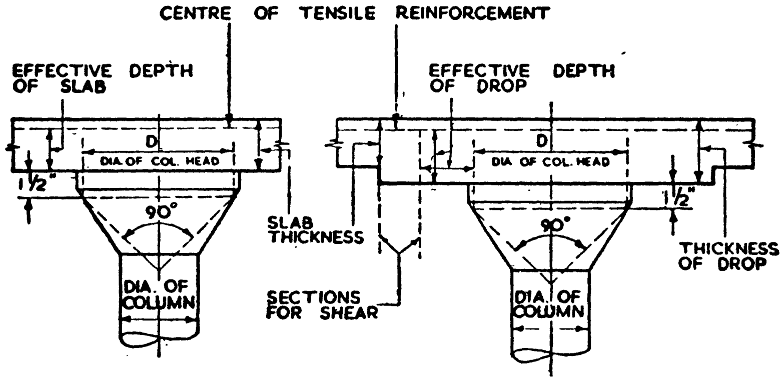
323. Columns supporting flat slabs. (*a*) *Column heads.* The heads of interior columns and such portions of the heads of exterior columns as will lie within the building should satisfy the following requirements :—

(i) The angle of greatest slope of the column head should not exceed 45 degrees from the vertical.

(ii) For the purpose of clause 316, the diameter of the column head, D, is its diameter at a distance of 1½ in. below the underside of the slab or the underside of the drop where provided, as shown in Fig. 6.

(iii) The diameter D should not be less than 0·2 L nor more than 0·25 L.

Where the column and column head are not of circular cross-section the term 'diameter' used above should be deemed to mean the diameter of the largest circle which can be drawn within the section.



(*a*) Panels without drop. (*b*) Panels with drop.

FIG. 6.—DIAGRAMS SHOWING THE DETERMINATION OF THE DIAMETER D OF THE COLUMN HEAD FOR THE PURPOSE OF CLAUSE 316.

(*b*) *Bending moments in columns.* (i) Moments in internal and external columns should be provided for and should be taken as equal to 50 and 90 per cent respectively of the negative moment in the column strip specified in clause 316 (*a*). This moment should be apportioned between the upper and lower columns in proportion to their stiffnesses.

(ii) In the case of external columns carrying portions of the floors and walls as a cantilevered load, the specified column moments may be reduced by the moment due to the dead load on the cantilevered portion.

In determining the stiffnesses of the columns so as to apportion the specified bending moments, it would be necessary, theoretically, to make allowance for the added stiffness given by the splayed head. Such a refinement is, however, unnecessary since the specified bending moments take no account of the relative stiffness of slab and columns and are, therefore, only a very rough approximation to the true values. A sufficiently exact estimate of the stiffness will be obtained by considering the height of the column as extending to half-way up the splayed head.

WALLS

324. Reinforced concrete walls. Where reinforced concrete walls are intended to carry vertical loads, they should be designed generally in accordance with the recommendations given for columns. The cross-sectional area of the vertical reinforcement may, however, be reduced to not less than 0·4 per cent, and the lateral reinforcement parallel to the wall face to not less than 0·2 per cent. The provisions of clause 311 (*b*) with regard to lateral reinforcement at right angles to the wall face need not be taken to apply to walls in which the concrete stresses do not exceed 75 per cent of the permissible stresses specified in clause 303, and for such walls the minimum percentages of vertical and lateral reinforcement may be halved.

In load-bearing concrete walls* reinforcement may be necessary to control shrinkage cracking, or to strengthen the wall against bending. It is rarely required for increasing the direct load that the wall can carry. Reinforcement may sometimes be completely omitted from internal load-bearing walls, in which case the provisions of Clause 324 apply only to the reinforced concrete external walls. The reduced quantities of reinforcement allowed by this clause, as compared with those for columns in Clause 311, are little more than would be introduced to control cracking due to shrinkage or thermal movements. This is particularly the case when a low stress is adopted for the concrete, and the design is essentially that for a wall of plain concrete.

Transverse reinforcement is necessary for reinforced concrete walls (except where the reduced stresses in the concrete are used) and may be in the form of separate ties between pairs of bars near opposite faces of the wall or of continuous horizontal longitudinal zig-zag bars anchoring the vertical bars alternately near opposite faces.

FIRE RESISTANCE

325. Fire resistance of reinforced concrete. Consideration should be given to the fire resistance of reinforced concrete members to see that they provide the grade of fire resistance for the particular occupancy and size of a building or compartment as laid down in the Code of Functional Requirements, Chapter IV, 'Precaution against fire.'

The nature of the aggregates used in concrete has an important bearing on the resistance against fire, and for practical purposes they may be considered in two broad groups. The first group (called Class 1 aggregates) includes foamed slag, pumice, blastfurnace slag, crushed brick and burnt clay products, well-burnt clinker, and crushed limestone ; these aggregates show very good behaviour at high temperatures. The second group (called Class 2 aggregates) behave less well and consist of siliceous aggregates, e.g. flint, gravel, granite and all crushed natural stones other than limestone.

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* Reference should also be made to B.S. Code CP 111 (1948) and sub-code 111.201

In the case of small reinforced concrete columns, from 10 to 12 in. across, the use of a Class 1 aggregate is desirable if a fire grading of two hours is required. If only one hour's resistance is necessary, the Class 2 aggregate is satisfactory. A two-hours' resistance can be obtained with Class 2 aggregates if a light mesh reinforcement is placed centrally in the concrete cover to the main reinforcement.

For larger columns, a fire resistance of two hours is obtained with any aggregate. A four-hour resistance can be obtained, if required, by the use of Class 1 aggregates or a mesh placed centrally in the concrete cover to the main reinforcement.

The thickness of walls and floors which will give the various grades of fire resistance are shown in Table 14.

TABLE 14.—THICKNESS OF WALLS AND FLOORS FOR FIRE-RESISTING PURPOSES.

Grade of fire resistance	6 hours	4 hours	2 hours	1 hour	½ hour
Thickness of wall to attain grade :					
With Class I aggregates	8 in.	6 in.	4 in.	3 in.	3 in.
With Class II aggregates	9 in.	7 in.	4 in.	3 in.	3 in.
Thickness of solid reinforced concrete slab to attain grade	7 in.	6 in.	5 in.	4 in.	3½ in.
Thickness of concrete slab and of solid material in tiles of hollow tile floor to attain grade	6 in.	5 in.	3½ in.	3 in.	2½ in.

For hollow tile floors, the values given in the table assume a minimum cover to the steel of 1 in. for the four-hour grade, ¾ in. for the two- or one-hour grade, and ½ in. for the half-hour grade.

It will be seen from the figures above that the thicknesses used for structural reasons will normally lead automatically to a sufficient degree of fire resistance.

Data on the fire resistance of reinforced concrete are not so extensive as desired, and the information in this clause is based on a limited number of fire tests and on practical experience in actual fires.

The fire resistance of columns is considerably dependent on whether the cover of concrete spalls and falls away from the reinforced core. The provision of light mesh reinforcement centrally in the cover of concrete may sometimes, by preventing this concrete from falling away, double the fire resistance of the column.

It should be noted that the effect of intense fire on some steels with increased yield strengths obtained by cold-working is to reduce the steel to the same condition as ordinary mild steel. The proportionate loss of strength of the reinforced concrete member is thus greater than when mild steel is used. Where fire resistance is an important feature of the design, it may be advisable to avoid the use of such types of steel.

SECTION 4.—WORK OFF SITE

401. General. For reinforced concrete work, many operations can be performed equally well on or off the site and the choice is purely a matter of convenience. In this code, therefore, all workmanship is dealt with under Section 5.

SECTION 5.—WORKMANSHIP

501. Concrete.

(*a*) MEASURING. The quantity of cement should be determined by weight. The quantities of fine and coarse aggregates should be determined either by volume or preferably by weight. In the latter case, the weight should be determined from the volume given in Tables 1 or 2, and the density of the aggregate. The proportions given in clause 206 are based on the assumption that the aggregates are dry. If aggregates are used wet, due allowance must be made for bulking. See clause 805.

Wherever practicable, the materials for concrete should be proportioned by weight. Measurement of aggregates by volume can lead to considerable variation in the proportions from those assumed in the design, or used for the preliminary tests.

(*b*) MIXING. The concrete should be mixed in an approved mechanical batch mixer.

Mixing should continue until there is a uniform distribution of the materials and the mass is uniform in colour and consistence, but in no case should the mixing continue for less than two minutes.

The time of mixing required to produce good concrete varies with the type and size of mixer used. The specified minimum of two minutes will in most cases be sufficient.

(*c*) TRANSPORTING. Concrete should be handled from the place of mixing to the place of final deposit as rapidly as practicable by methods which will prevent the segregation or loss of the ingredients. It should be deposited as nearly as practicable in its final position to avoid rehandling or flowing.

The principal matters requiring attention in examining the efficiency of a method of transportation are whether or not it will produce segregation of the ingredients of the concrete, and whether the method is such that excessive water or unfavourable grading of the aggregate is necessary for its operation. The time taken in transporting should be the shortest practicable. During prolonged transportation the concrete may tend to stiffen and for use would require re-mixing with a further addition of water. This practice should be prohibited.

(*d*) PLACING.

(i) *General.* The concrete should be placed before setting has commenced and should not be subsequently disturbed.

(ii) *Construction joints.* Concreting should be carried out continuously up to construction joints, the position and arrangement of which should be predetermined by the designer.

When work has to be resumed on a surface which has hardened, such surface should be removed by hacking. The surface should then be swept clean, thoroughly wetted, and covered with a ½ in. layer of mortar composed of cement and sand in the same ratio as the cement and sand in the concrete mixture. This ½ in. layer of mortar should be freshly mixed and placed immediately before the placing of the concrete.

(iii) *Compacting.* Concrete should be thoroughly compacted during the operation of placing, and thoroughly worked around the reinforcement, around embedded fixtures and into corners of the formwork.

(iv) *Mechanical vibration.* The use of mechanical vibrators for compacting concrete is strongly recommended, provided that the reduced water/cement ratios recommended in clause 206 are adopted. It is also recommended that where it is to be adopted, the reinforced concrete designer should include a specification for this.

In normal weather a period up to half an hour between the addition of water to the materials and the final placing of the concrete may be allowed. In cold weather this time may be extended, but in hot weather the period should be shorter. Cements, however, do not all behave in the same manner and with some the periods may be increased.

Construction Joints.—The treatment of concrete at construction joints is a matter of considerable importance. Where trouble has occurred in reinforced concrete work through the use of inferior concrete, experience shows it to be much more pronounced at these joints than elsewhere in the structure. In making horizontal joints it is found that the weakness may occur either at the top of the concrete at the underside of the joint, or at the bottom of the concrete at the upper side of the joint. The trouble at the top of the concrete arises from the formation of laitance, a kind of scum that comes from the cement, on the surface. This is more noticeable with very wet concretes, particularly where finely-ground cements are used. As a result the top layer of concrete is excessively weak and permeable, and has little power to resist destructive agencies such as frost and corrosive waters. At the upper side of the joint troubles principally arise from an excess of aggregate and a deficiency of cement due to segregation.

Laitance formed on the surface at a construction joint should preferably be removed before the concrete has hardened, care being taken not to disturb the concrete sufficiently to leave it in a porous and weakened condition. Above all it is important to see that set concrete is thoroughly clean before work is resumed on it.

Vertical joints, while not liable to the defects possible in horizontal joints, are more prone to shrinkage if the joint is weaker than the concrete elsewhere. Vertical stop-boards, whether in beams, slabs, or walls, should always be provided at the ends of each section of work to be concreted in one operation. These boards provide a surface against which the concrete can be properly tamped and consolidated, whereas if the concrete flows at an angle the cement runs along the shuttering and is wasted, the concrete being invariably porous and deficient in cement.

All construction joints in walls and slabs can with advantage be rebated or joggled as shown in *Fig.* 18A, the shape of the rebate or joggle being altered as required. The increased resistance to the percolation of moisture from one side to the other of joints formed in this way is well worth the small extra cost.

The appearance of construction joints, so often unsightly on walls and elsewhere, can be greatly improved (and in fact they can be made almost invisible) by attaching a small fillet to the shuttering, as shown in *Fig.* 19A. This fillet can be removed immediately the concrete has set.

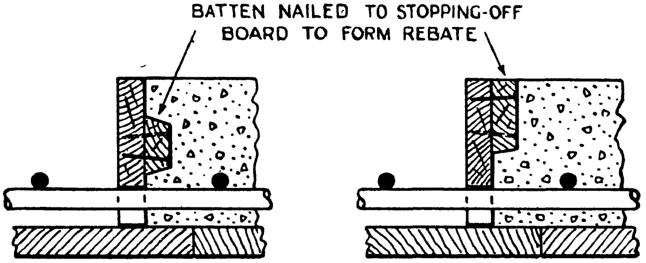

FIG. 18A.—ALTERNATIVE METHODS.

Compacting.—The operation of compacting is of fundamental importance in the production of good concrete. It is necessary to ensure that a thoroughly dense concrete is produced and that all reinforcement bars are entirely coated with cement. It should be borne in mind that the strength tests specified in this Code are based on ramming conditions adequate to produce practically the maximum density and strength that can be obtained with concrete of the degree of workability usually employed in reinforced concrete work. This is important, since it must be realised that unless the concrete is properly consolidated in the structure its strength will fall short of the values determined by the works tests.

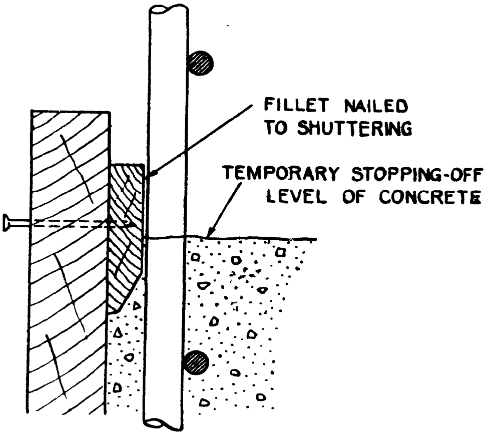
FILLET NAILED TO SHUTTERING
TEMPORARY STOPPING-OFF LEVEL OF CONCRETE

FIG. 19A.

MECHANICAL VIBRATION. The application of vibration to in-situ concrete has been accompanied by many difficulties and, although the frequencies and accelerations necessary to consolidate precast products are known and can easily be applied, it is not possible to specify them precisely for concrete placed in situ since the laws of transmission of vibration through concrete are not yet fully understood. It can be said, however, that as a general rule the frequencies should not be less than 3,000 complete cycles (or vibrations) per minute. While known accelerations can be applied to the shuttering of in-situ concrete it is not easy to determine the resultant accelerations in the concrete itself, and the successful use of vibrators must therefore depend upon practical experience in applying them. Accelerations of four times the acceleration due to gravity, as used for precast products, are usually desirable if full advantage is to be obtained from the reduced water content possible with vibrated concrete.

Where vibration is applied externally there is difficulty in providing shuttering which is sufficiently rigid to resist deformation and at the same time flexible enough to transmit efficiently the vibrations to the concrete. Special types of flexible joints have been developed which enable shuttering to be vibrated efficiently, and to remain watertight, without affecting concrete already cast.

With cast-in-situ concrete one of the main problems is to decide how long to vibrate the concrete. If the vibration is unduly prolonged segregation of the aggregate may occur, and the likelihood of this is increased where unsuitable mixes—usually too wet—are employed. Too short a period of vibration leaves a honeycombed surface and, since a suitable period can only be determined by trial, it is recommended that the advice of the makers of the machinery be sought. As a general guide it can be said that vibration should be stopped when air bubbles have ceased, or practically ceased, coming to the surface, and the surface itself is continuous.

(*e*) CURING. The concrete should be covered with a layer of sacking, canvas, hessian or similar absorbent material, and kept constantly wet for seven days, except in the case of high-alumina cement when this period may be reduced to twenty-four hours. Alternatively, the concrete being thoroughly wetted, may be covered by a layer of approved waterproof material which should be kept in contact with it for seven days.

Curing is particularly important with cements that harden rapidly. It is recommended by the makers of high-alumina cement that the shuttering should be thoroughly wetted and should normally be struck at the age of eight hours, and the concrete kept wet for at least 24 hours; otherwise the strength is much impaired and objectionable surface dusting may occur. The water also serves to maintain a low temperature in the concrete, which in large masses of high-alumina cement concrete may rise sufficiently to cause deterioration in strength.

(*f*) WORK IN COLD WEATHER. When depositing concrete at or near freezing temperatures, precautions should be taken to ensure that the concrete has a temperature of at least 40° F and that the temperature of the concrete is maintained above 36° F until it has thoroughly hardened. When necessary, concrete materials should be heated before mixing and carefully protected after placing. Dependence should not be placed on salt or other chemicals for the prevention of freezing. Calcium chloride may be used to accelerate the rate of hardening, usually 1½ per cent by weight of the cement will prove sufficient and there are dangers associated with an excess. No frozen material or materials containing ice should be used.

All concrete damaged by frost should be removed.

The rate of hardening of concrete is appreciably affected by its temperature. An indication of the effect may be seen in *Table* 35A which gives typical strengths from the results of tests.* These figures are based on average strengths from a number of cements, the order of the range of strengths between cements from different sources being as indicated in *Fig.* 1A. At low temperatures both ordinary and rapid-hardening Portland cements show a very marked diminution in rate of hardening. Taking the strength of ordinary Portland cement concrete at 28 days as 100 per cent. as given in the table, it will be seen that only 10 per cent. of this strength is attained in three days at low temperatures, 37 per cent. at normal temperatures, and 59 per cent. at high temperatures. The reduction, it should be noted, is only in the rate of hardening, the strength at 28 days and later being only slightly affected.

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* Building Research Technical Paper No. 14, by N. Davey.

Concrete which is frozen after hardening is not permanently injured. The hardening is delayed by the period during which it has been frozen.

The setting and hardening of cement are accompanied by the evolution of heat. In small masses of the size of the cubes used in obtaining the figures given in *Table* 35A, this heat is rapidly dissipated and the temperature of the concrete remains sensibly the same as that of the surrounding air. In large masses of concrete the heat is not so readily dissipated, and the higher temperature results in improved rates of hardening. The effect is large in members measuring more than 2 ft. 6 in. in each direction, in which the temperature rise may be about 50 deg. F. and may persist for some days. Referring to *Table* 35A it will be seen that this would be sufficient to improve the rate of hardening, for example, from low-temperature conditions to between normal and high.

TABLE 35A.—EFFECT OF TEMPERATURE ON THE STRENGTH OF CONCRETE.

Type of cement	Age (days)	Crushing strength of 1 : 2 : 4 concrete (expressed as a percentage of 28-day strength of ordinary Portland cement concrete)		
		Low temperature (36 deg. F.)	Normal temperature (64 deg. F.)	High temperature (95 deg. F.)
Ordinary Portland cement	1	0	10	35
	3	10	37	59
	7	25	64	77
	14	55	80	90
	28	92	**100**	102
	56	108	110	110
Rapid-hardening Portland cement	1	0	20	50
	3	12	59	82
	7	32	93	102
	14	70	107	120
	28	118	126	128
	56	135	135	135
High-alumina cement	1	54	146	108
	3	164	188	131
	7	188	197	66
	14	197	212	60
	28	207	218	55
	56	222	229	55

The effect of low temperature on the rate of hardening of high-alumina cement concrete is not nearly so pronounced as in the case of Portland cement concrete. This is due partly to the fact that the setting and hardening of this type of cement are accompanied by the evolution of much more heat. There is, however, an important effect at high temperatures. Above 80 deg. F. there is a marked falling off in strength; the strength at this temperature may be only 25 per cent. of the strength at normal temperatures. For this reason it is important that in no circumstances should high-alumina cement concrete be placed at high temperatures, or in masses of such dimensions that high temperatures will be developed by the evolution of heat.

It is useful in practice to know the size of mass for which it is safe to rely on the heat evolved by the cement as a means of preventing trouble from frost. As an indication of this the figures given in *Table* 36A have been calculated from a theory which has been substantiated by experiment.* In calculating these figures it has been assumed that the air temperature at the time of placing the concrete is 40 deg. F., and that in the next twelve hours it falls at a uniform rate to 28 deg. F. and then remains steady. These conditions are arbitrary, but may be regarded as representing a typical frosty period in Britain. The calculated figures will not apply to conditions of very severe frost. The dimensions given indicate masses of concrete that, in the conditions already mentioned, would not fall below freezing point until at least seventy-two hours after placing. In these conditions it is thought that the concrete will have hardened sufficiently to ensure that no damage will result from frost. Figures are included for a column and for a slab, and in both cases two conditions are given, one in which the materials are mixed at air temperature and the other in which they are preheated to a temperature of 58 deg. F. This temperature may be regarded as the practical limit to which the concrete can be raised by preheating materials in frosty weather. The figures given will not vary greatly for any of the different concrete aggregates generally in use. The figures for the 1 : 3 : 6 mixes have been included in the

TABLE 36A.—MINIMUM THICKNESS OF CONCRETE THAT SHOULD BE PLACED IN FROSTY WEATHER. (For conditions, see text.)

Type of cement	Proportions (by weight)	Water-cement ratio (by weight)	Minimum diameter of columns (in.)		Minimum thickness of slabs (in.)	
			Concrete at air temperature of 40 deg. F.	Concrete preheated to 58 deg. F.	Concrete at air temperature of 40 deg. F.	Concrete preheated to 58 deg. F.
Ordinary Portland	1 : 1 : 2	0·45	20	9	10	4½
	1 : 2 : 4	0·60	31	14	17	7½
	1 : 3 : 6	0·75	46	19	25	10
Rapid-hardening Portland	1 : 1 : 2	0·45	13	7	7	3½
	1 : 2 : 4	1·60	22	10	12	5
	1 : 3 : 6	0·75	29	13	16	7

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* Building Research Technical Paper No. 15, by N. Davey and E. N. Fox.

table as of interest for comparison with the figures for other mixes, although it is realised that such mixes are not generally used.

The Use of Calcium Chloride.—When calcium chloride is added to concrete, it increases the rate of chemical reaction, thus leading to (i) an increased rate of evolution of heat during the setting of the cement and the early period of hardening and (ii) an increased rate of development of strength during this period. Consequently the use of calcium chloride can be of great value for concrete work in cold weather.

An excessive amount of calcium chloride may cause too rapid setting, leading to difficulty in placing the concrete, and may also result in corrosion of the reinforcement. A limit of 2 per cent. of calcium chloride by weight of the cement should not be exceeded.

Calcium chloride should not be used with high-alumina cement.

502. Steel. (*a*) *Cleaning.* All metal for reinforcement should be free from loose mill scale, loose rust, oil and grease, or other harmful matter, immediately before placing the concrete.

(*b*) *Placing.* All reinforcement should be placed and maintained in the position shown on the drawings. Many otherwise good reinforced concrete structures have given trouble after some years of weathering by reason of some of the reinforcements being too close to the surface with the result that corrosion of the reinforcement has taken place. Some definite method of ensuring the adequacy of cover should be sought by the contractor and the designer acting in conjunction.

(*c*) *Bending.* Reinforcement should not be bent or straightened in a manner that will injure the material.

Bending hot at a cherry-red heat (not exceeding 1550° F) may be allowed except for bars which depend for their strength on cold working. Bars bent hot should not be cooled by quenching.

(*d*) *Welding.* Welding by gas or electricity may be permitted under suitable conditions and with suitable safeguards, and in accordance with B.S. 693, 'Oxy-acetylene welding in mild steel' and B.S. 538, 'Metal arc welding in mild steel.'

Welding is of two kinds : (i) Tack or positional welding between rods crossing more or less at right angles so as to fix them in position. (ii) Butt welding between the ends of rods in line whereby stress is transferred across the section. In the case of (i) no special precautions in regard to stress are needed. In the case of (ii) it is recommended that this should only, at present, be allowed for mild steel. In the case of rods of mild steel which have their strength increased by cold working, butt welding may be permitted, but the stress at the weld should be limited to that appropriate to mild steel, and the additional strength obtained by cold working should be ignored at and near the weld.

503. Formwork. (*a*) *General.* The formwork should be so constructed as to remain sufficiently rigid during the placing of the concrete and should be sufficiently tight to prevent loss of liquid from the concrete.

Attention is drawn to the importance of the design of the formwork which can be an important factor in the economy and efficiency of reinforced concrete construction. The detailed design should preferably be undertaken by an expert and is considered to be outside the scope of the present code. The important qualities of good formwork relate to: rigidity and strength, economy of material, re-use, convenience of erection and striking, and good surface finish.

(*b*) *Strutting.* The vertical strutting should be carried down to such construction as is sufficiently strong to afford the required support without injury.

(*c*) *Cleaning and treatment of forms.* All rubbish, particularly chippings, shavings and sawdust, should be removed from the interior of the forms before the concrete is placed and the formwork in contact with the concrete should be cleaned and thoroughly wetted or treated with an approved composition. Care should be taken that such approved composition is kept out of contact with the reinforcement.

(*d*) *Stripping time.* In no circumstances should forms be struck until the concrete reaches a cube strength of at least twice the stress to which the concrete may be subjected at the time of striking.

The cube strength referred to should be that of concrete using the same cement and aggregates and proportions and cured at the same temperatures as the work. Where there is no great objection, the formwork should be left up longer, as assisting the curing.

(*e*) *Procedure when striking.* All formwork should be removed without such shock or vibration as would damage the reinforced concrete. Before the soffit and struts are removed the concrete surface should be exposed where necessary, in order to ascertain that the concrete has sufficiently hardened. Proper precautions should be taken to allow for the decrease in rate of hardening that occurs with all cements in cold weather.

(*f*) *Camber.* It is generally desirable to give forms an upward camber to ensure that the beams will not have a sag when they have taken up their deflection, but this should not be done unless allowed for in the design calculations of the beams.

(*g*) *Tolerances.* Formwork should be so constructed that the internal dimensions are within the permissible tolerances specified by the designer.

Compositions for painting on formwork may consist of mineral oils (some of which are sold as mould oils), limewash, or a form of retarder which delays the

TABLE 37A.

Removal of shuttering	Ordinary Portland cement concrete		Rapid-hardening Portland cement concrete	
	Cold weather (about freezing)	Normal weather (about 60 deg. F.)	Cold weather (about freezing)	Normal weather (about 60 deg. F.)
	Days	Days	Days	Days
Beam sides, walls, and columns (unloaded)	8	3	8	2
Slabs (props left under)	10	4	10	3
Removal of props to slabs . . .	14	10	14	5
Beam soffits (props left under) . .	12	8	12	5
Removal of props to beams . . .	28	21	21	8

Note.—For high-alumina cement concrete the whole of the formwork may be removed after the concrete has been in place for 24 hours.

setting of the cement and on removal of the forms allows the surface skin of cement to be brushed away to expose the aggregate for the purpose of obtaining a desired finish.

PROCEDURE WHEN STRIKING FORMS.—Suggested minimum intervals of time between the placing of the concrete and the striking of the shuttering are given in *Table* 37A, the periods being based on cements of average qualities. The number of days during which the temperature has fallen below freezing point should be added to these times.

504. Fixing blocks. Fixing blocks may be embedded in the concrete provided that the strength or effective cover of any part of the structure is not reduced below the standard required by this code.

505. Electric currents. No part of the reinforcement should be used for conducting electric currents.

SECTION 6.—INSPECTION AND TESTING

601. Sampling. If samples of materials proposed for the work are required for testing, they should be submitted in sufficient time for the tests to be made before the materials in bulk are delivered to the site.

602. Supervision. It is exceedingly difficult and costly to alter concrete work once placed, hence constant and strict supervision of all the items of the construction is necessary during the progress of the work, including the proportioning and mixing of the concrete. Supervision is also of extreme importance to check any reinforcement and its placing before being covered.

Before any important operation, such as concreting or stripping of the formwork, is begun, adequate notice should be given to the construction supervisor.

One of the basic assumptions made in drafting the Code (see Clause 101), and in fixing the permissible stresses, is that the work is carried out under the direction of a qualified supervisor.

603. Testing of materials. The concrete should be tested from time to time as the work proceeds for consistency and strength by the methods described in Section 8.

Periodical tests should be made to ensure that the aggregates satisfy the requirements of the appropriate British Standards or other specifications, and that the reinforcing bars have a yield strength not less than that assumed in the design.

604. Inspection. After stripping the formwork, the concrete should be carefully inspected to see that it is free from defects before the concrete has attained its maximum hardness.

605. Load testing of structures. Where there is a doubt as to the strength of a structure, a test load may be applied. This test load should be 125 per cent of the original superimposed load for which the structure was, or should have been, designed. Such test load should not be applied before 28 days after the date of concreting. During the test, struts strong enough to take the whole load should be placed in position leaving a gap under the members. If, under test, no structural defects can be observed and the deflection is not excessive, the structure should be deemed to be satisfactory.

In carrying out loading tests and interpreting the results, it is necessary carefully to consider the weather conditions that were prevalent during the hardening period of the doubtful work. A generous hardening period should always be allowed, considerably longer being necessary in cold than in normal or warm weather. Reference to *Table* 35A will enable an estimate of the effect of weather conditions to be made, but in any case it is extremely unlikely that a longer period than 56 days will ever be required. Tests should normally be made at an age of four to six weeks. Testing before 28 days from the date of concreting is not allowed by Clause 605, to avoid possible permanent weakening of the structure.

No specific test requirements, other than the test load, are given in the Code. The commonest tests in practice are on floors and for these it is usual to maintain the load for 24 hours and to expect a recovery in deflection, on removal of the load, of at least 75 per cent. of the maximum deflection shown during the 24 hours under load.

During the 24 hours under the test load every reinforced concrete structure will show a progressively increasing deflection. This is principally due to the creep of the concrete under load (see comment on Clause 301). The effect is particularly noticeable when low-strength concrete is used, and if its strength is 2250 lb. per square inch, for example, the progressive deflection may almost equal the initial elastic deflection. Members constructed of such concrete, even though sound, may not show a recovery of 75 per cent. of the maximum deflection immediately upon removal of the load. Some subsequent recovery should take place, however, bringing it within the prescribed limits. If within two or three days this is not the case, a second test should be made.

Creep will be less during the second test, and in a sound structure no difficulty is likely to occur in conforming to the recovery of 75 per cent. required. Usually in unsound members evidence of weakness will be apparent in other ways, generally in the form of excessive cracking. Fine hair cracks (ordinarily unnoticed) do not indicate weakness or failure, and with an overload of 25 per cent. may be expected.

Test loads in excess of one-and-a-quarter times the specified superimposed load are undesirable since it is necessary to avoid the risk of over-straining any part of the work. It must be noted that the test load, in addition to the superimposed load, must include all dead weight (not at that time applied) such as floor finish and partitions. Also it is desirable to make a record of temperatures throughout the testing period.

SECTION 7.—MAINTENANCE

701. General. No structural maintenance should be necessary for dense concrete constructed in accordance with this code. However, it is recommended that a periodical check should be made (e.g. every three or five years) to detect any excessive cracking or other defect which should receive attention. Where corrosion of the bars has caused staining or has loosened the concrete cover, exposing, cleaning and re-covering the bars will add considerably to the life of the structure. The cost of such work will be small compared with that necessary to repair the structure if the rusting had been allowed to develop over a long period of time.

Where the cost is not prohibitive, the use of a good concrete paint will often enhance the appearance of the structure, as well as giving added protection against corrosion. The paint used should, however, be suitable for the alkaline conditions of concrete work. Suitable types of paint and the precautions to be taken in their use are dealt with in Code 231.403, 'Painting and distempering Portland cement concrete and cement plaster.'

Too many reinforced concrete structures have, after only twenty years or so, become unsightly and possibly unsafe because of the corrosion of the reinforcement. This condition has arisen chiefly from local patches of poor permeable concrete and from inadequate cover over the reinforcement. The early signs of corrosion can be observed if careful periodical inspections of the structure are made, and suitable measures can then be taken to prevent further corrosion before it has developed to the stage where extensive spalling of the concrete occurs.

The painting of concrete requires special precautions because of two conditions which readily lead to failure of the film of paint. These are (i) the concrete contains free moisture which affects the adhesion between the paint and the concrete causing peeling and blistering, (ii) the caustic alkalis present in the cement attack the film of paint and may cause bleaching. The painting of concrete should therefore be delayed as long as possible so that it has dried out appreciably, and by surface carbonation reduced the danger of alkali attack. Generally, it is best to use one of the special cement paints, which are not attacked by alkalis and also have sufficient porosity to allow the concrete to continue to dry out after painting.

SECTION 8.—APPENDICES

801. Standard method of test for consistence of concrete (slump test). This test may be used during the progress of the work for ensuring uniformity in the consistence of concrete.

The test specimen should be formed in a mould in the form of the frustrum of a cone with internal dimensions as follows : bottom diameter, 8 in. ; top diameter, 4 in. ; height, 12 in. The bottom and the top should be open, parallel to each other and at right angles to the axis of the cone. The mould should be provided with suitable footpieces and handles. The internal surface should be smooth.

Care should be taken to ensure that a representative sample is taken.

The internal surface of the mould should be thoroughly clean, dry and free from set cement before commencing the test.

The mould should be placed on a smooth, flat, non-absorbent surface, and the operator should hold the mould firmly in place, while it is being filled, by standing on the foot-pieces. The mould should be filled to about one-fourth of its height with the concrete which should then be puddled, using 25 strokes of $\frac{5}{8}$ in. diameter steel rod, 2 ft. long, bullet-pointed at the lower end. The filling should be completed in successive layers similar to the first and the top struck off so that the mould is exactly filled. The mould should then be removed by raising vertically, immediately after filling. The moulded concrete should then be allowed to subside and the height of the specimen measured after coming to rest.

The consistence should be recorded in terms of inches of subsidence of the specimen during the test, which is known as the slump.

Although the slump test, where carefully applied, is a means of maintaining reasonably uniform consistence with any given aggregate and cement, it is not a true measure of workability, and where materials are changed it may be necessary to change the slump required. When puddling the concrete in the cone, excessive vigour should not be used, and the cone should be lifted slowly and truly vertically.

It is desirable to carry out slump tests whenever preliminary or works cubes are made. In the case of works tests it is particularly important that the results of these tests should accompany the cubes when forwarded to the testing laboratory, as they are frequently of service in interpreting the results of the tests of the cubes. Provided that care is taken to ensure that no water is lost, the material used for the slump tests may be re-mixed with the remainder of the concrete before making the cubes.

802. Preliminary cube tests. Method to be used for compression tests on concrete in a laboratory where accurate control of materials and test conditions is possible :

(*a*) *The concrete.* The materials and proportions used in making the test specimens, including the water content, should be similar in all respects to those to be used in the work. The cement on arrival at the laboratory should be mixed dry either by hand or in a suitable mixer so as to ensure uniformity, care being taken to avoid the intrusion of foreign matter, and it should then be stored in air-tight containers until required. All materials should be brought to a temperature of 58° F to 64° F before beginning the tests, and the aggregates should be dry. The quantities of cement, aggregate and water for each batch should be determined by weight to an accuracy of 1 part in 1000.

The concrete should be mixed by hand or in a small batch mixer in such a manner as to avoid loss of water. If the concrete is mixed by hand, the cement and fine aggregate should first be mixed dry until the mixture is uniform in colour. The coarse aggregate should then be added and mixed with the cement and fine aggregate. The water should then be added and the whole mixed thoroughly until the resulting concrete is uniform in colour, and in no case for less than two minutes. If a batch mixer is used, all materials may be placed together in the mixer and mixed thoroughly until the resulting concrete is uniform in colour, and in no case for less than two minutes.

(*b*) *The test specimens.* The test specimens should be 6 in. or 4 in. cubes. Four-inch cubes should not be used when the maximum size of the aggregate exceeds ¾ in.

The mould should be of metal with inner faces accurately machined in order that the opposite sides of the specimen are plane and parallel. Each mould should be provided with a metal base having a smooth machined surface. The interior surfaces of the mould and base should be lightly oiled before concrete is placed in the mould.

Test specimens should be moulded by placing the fresh concrete in the mould in 2 in. layers, each layer being thoroughly compacted with a steel bar 15 in. long and having a ramming face 1 in. square and weighing 4 lb. At least 25 strokes of the bar should be given for each layer, although 35 or more may be necessary for dry mixes in 6 in. cubes.

Where it is proposed to use mechanical vibrators for compacting the concrete, and to allow increased stresses in accordance with clause 303 (*a*) (iii), the test specimens may be compacted with a mechanical vibrator or by hand.

(*c*) *Storage of test specimens.* Test specimens should be placed in moist air of at least 90 per cent relative humidity and at a temperature of 58° F to 64° F for 24 hours (± ½ hour), commencing immediately after moulding is completed. After 24 hours the test specimens should be marked, removed from the moulds and placed in water at a temperature of 58° F to 64° F until required for test.

(*d*) *Method of testing.* The tests should be made at the age of the concrete corresponding to that for which the strengths are specified in Tables 1 and 2. Compression tests should be made between smooth plane steel plates, without end packing, and a load should be applied axially at the rate of approximately 2000 lb. per sq. in. per minute. One compression plate of the testing machine should be provided with a ball seating in the form of a portion of a sphere, the centre of which coincides with the central point of the face of the plate. Test specimens should be placed in the machine in such a manner that the load is applied to the sides of the specimens as cast.

(*e*) *Standard of acceptance.* Three test specimens should be made for each age at which tests are required. The compressive strength should be calculated in lb. per sq. in. from the maximum load sustained by the cube before failure. The average of the strengths of the three specimens may be accepted as the compressive strength of the concrete provided that the difference between the maximum and minimum strengths of the three specimens does not exceed 15 per cent of the average strength. If the difference exceeds 15 per cent of the average strength, repeat tests should be made, unless the minimum strength is greater than the strength specified in Tables 1 or 2 as the case may be.

It should be fully realised that the preliminary tests must be carried out under properly-controlled conditions, and unless fully-competent operators are employed it may be difficult to comply with the requirement for standard of acceptance.

For preliminary tests the quantities of the materials are to be determined by weight. On the work the cement has to be used in weighed quantities, but the aggregates may be determined by volume. The basis of the proportions of the aggregates may therefore be one of volume on the work and of weight for the preliminary tests. Consequently in making preliminary tests it may be necessary first to determine the weights per unit volume of the aggregates. For this purpose the weight of aggregate required to fill a container of known volume must be ascertained. This should be done with the aggregate in a dry condition, the aggregate being well shaken down. In this determination errors outside the accuracy demanded for the measurement of the weight, namely one part in one thousand, will be introduced as a result of the difficulty of obtaining a standard amount of compaction of the aggregate. However, since the strength is mainly dependent on the ratio of the water to the cement, both of which can be measured accurately, the errors should not appreciably affect the strength of the concrete obtained.

803. Works cube tests. Method to be used for compression tests of concrete sampled during the progress of the work.

(*a*) *Sampling the concrete.* Concrete for the test specimens should be taken at the point of deposit. To ensure that the specimens are representative of the concrete in the structure a number of samples should be taken from different points. Each sample should be large enough to make one test specimen and should be taken from one point in the work. The location from which each sample is taken should be noted.

(*b*) *The test specimens.* The test specimens should be cubes of the size, and made in the manner prescribed in clause 802 (*b*).

(*c*) *Storage of test specimens.* The test specimens should be stored at the site at a place free from vibration, under damp sacks for 24 hours (± ½ hour), after which time they should be removed from the moulds, marked and stored in water preferably at a temperature of 50° F to 70° F until the date of test. Where storage in water is difficult, cubes may be stored in wet sand. Specimens which are to be sent to a laboratory for testing should be packed in damp sand, or other suitable damp material, for that purpose and should reach the laboratory at least 24 hours before the test. On arrival at the laboratory they should be similarly stored in water until the date of the test.

(*d*) *Method of testing and standard of acceptance.* The specimens should be tested in the manner prescribed in clause 802 (*d*) and the standard of acceptance should be as laid down in clause 802 (*e*).

In carrying out works tests it should be realised that a careful compliance with the requirements is of great importance in view of the higher stresses permitted and greater strengths required by the Code. Care is particularly necessary in sampling the concrete. The importance of obtaining a representative sample and ensuring that this is properly placed in the mould cannot be over-estimated.

The works test is one for quality of the concrete as deposited and does not necessarily represent the strength of the concrete in the member. The strength of the member will depend on its size, the amount of heat generated by the cement, and the temperature of the member during and after casting (see comment on

Clause 501), which may not correspond with the temperature of storage of the cubes. In estimating the strength of the member, allowance must be made for any difference. Also, in interpreting test results, attention must be given to the methods of consolidating the concrete in the member. Thus in large members the concrete may be practically self-consolidating, but in narrow and confined spaces it is often very difficult to obtain the same degree of compaction as that specified for the cubes.

Works cubes should be marked with the date of casting, and when sent to the laboratory they should be accompanied by particulars giving the proportions of the concrete and the position in the work from which the sample was taken.

804. Works beam tests. Method of making transverse tests of concrete sampled during the progress of the work.

(*a*) *The test specimens.* The test specimens should be 16 in. × 4 in. × 4 in. beams. (These dimensions are suitable only when the maximum size of the aggregate does not exceed $\frac{3}{4}$ in.)

Samples of concrete should be taken in the manner prescribed in clause 803 (*a*). Test specimens should be made as described in clause 802 (*b*), except that, in compacting, the number of strokes of the bar per layer of concrete should be at least 70. One hundred strokes may be necessary for dry mixes. The specimens should be stored as described in clause 803 (*c*).

(*b*) *Method of testing.* The test beam should be evenly supported upon two parallel steel rollers of $1\frac{1}{2}$ in. diameter, the distance between the centres of the rollers being 12 in. The load should then be centrally applied through a third steel roller also $1\frac{1}{2}$ in. diameter, placed midway between the supports. The length of all rollers should exceed the width of the beam. Test specimens should be placed in the machine in such a manner that the load is applied to the sides of the specimens as cast. The loading should be applied smoothly and gradually, at such a rate as will ensure that the maximum load is reached in approximately five minutes. The modulus of rupture (R) in lb./sq. in. should be calculated from the formula :

$$R = \frac{9}{32} W$$

where W is the total load in lb. at which the beam fails.

This formula only applies when the test beams are 16 in. × 4 in. × 4 in.

Considerable data have been collected on the transverse strength of concrete using the test described in Clause 804, and the small testing machine necessary can be made fairly cheaply and transported from one site to another. The total load required is usually less than 1 ton and this can be produced by a suitable system of levers, together with dead weights or a spring balance.

A further advantage of the size of test specimen suggested is that the halves of the broken specimen can be used subsequently for compression tests [see comment on Clause 208 (*c*)].

805. Field method of determining the necessary adjustment for the bulking of fine aggregate. (*a*) *Explanation and scope.* Sand brought on to a building or other works may contain an amount of moisture which will cause it, when loosely filled into a container, to occupy a larger volume than it would occupy if dry. If, as is usual, the sand is measured by loose volume, it is necessary in such a case to increase the measured volume of the sand, in order that the amount of sand put into the concrete may be the amount intended for the nominal mix used (based on dry sand). It will be necessary to increase the volume of sand by the 'percentage' bulking. The correction to be made is only a rough approximation, because the system of measurement by loose volume is a rough method at the best, but a correction of the right order can easily be determined and should be applied in order to keep the concrete uniform.

(*b*) *Procedure.* The procedure to be adopted may be varied, but two methods are suggested below. Both are dependent on the fact that the volume of inundated sand is the same as if the sand were dry.

(i) Put sufficient of the sand loosely into a container until the container is about two-thirds full. Level off the top of the sand and pushing a steel rule vertically down through the sand at the middle to the bottom, measure the height. Suppose this is h inches.

Empty the sand out of the container into another container where none of it will be lost. Half fill the first container with water. Put back about half the sand and rod it with a steel rod about $\frac{1}{4}$ in. in diameter, so that its volume is reduced to a minimum. Then add the remainder of the sand and rod it in the same way. Smooth and level the top surface of the inundated sand and measure its depth at the middle with the steel rule. Suppose this is h' inches. The percentage of bulking of the sand due to moisture should be calculated from the formula :

$$\text{Percentage bulking} = \left(\frac{h}{h'} - 1\right) \times 100$$

(ii) In a 250 c.c. measuring cylinder pour the damp sand (consolidated by shaking) until it reaches the 200 mark. Then fill the cylinder with water and stir the sand well. (The water must be sufficient completely to submerge the sand.) It will be seen that the sand surface is now below its original level. Suppose the surface is at the mark x c.c. The percentage of bulking of the sand due to moisture should be calculated from the formula :—

$$\text{Percentage bulking} = \left(\frac{200}{x} - 1\right) \times 100.$$

806. Schedule of symbols. The symbols used in this code are as follows :—

A = cross-sectional area of concrete excluding any finishing material or reinforcing steel.
A_b = equivalent area of helical reinforcement (volume of helix per unit length of column).
A_c = cross-sectional area of steel in compression.
A_k = cross-sectional area of concrete in column core, excluding area of longitudinal reinforcement.
A_w = cross-sectional area of stirrup.
a = lever arm of the resistance moment.
B = overall breadth of compressive flange of a beam.
B_1 = permissible average bond stress.
B_2 = permissible local bond stress.
b = breadth of a rectangular beam or the breadth of the rib of a T- or L-beam.
c = permissible stress in concrete in direct compression.
c_1 = permissible compression stress for column bars.
D = diameter of column head supporting flat slab. (It is elsewhere used for overall concrete depth, or least lateral dimension of a column.)

K_b = stiffness of beam.

K_{b1} = stiffness of beam on one side of a column.

K_{b2} = stiffness of beam on the other side of a column.

K_l = stiffness of lower column.

K_u = stiffness of upper column.

L = length of a column or beam between adequate lateral restraints. (In the case of slabs L is the average L_1 and L_2.)

L_1 = (in the case of flat slabs) length of panel in the direction of span.

L_2 = (in the case of flat slabs) width of panel at right angles to direction of span.

l = effective span of beam or slab or effective height of column.

l_x = length of shorter side of slab spanning in two directions.

l_y = length of longer side of slab spanning in two directions.

M = bendiing moment (suffixes as required).

M_e = bendng moment at end of beam framing into a column, assuming fixity at connection.

M_{es} = the maximum difference between the moments at the ends of two beams framing into opposite sides of a column, each calculated on the assumption that the ends of the beams are fixed and that one of the beams is not loaded.

M_x and M_y = maximum bending moments, for spans l_x and l_y respectively, on strips of unit width in slabs spanning in two directions.

m = modular ratio (15).

o = sum of perimeters of the bars in the tensile reinforcement.

P = axial load permissible on a short column.

p = pitch or spacing of stirrups.

R = modulus of rupture.

S = total shear across a section.

s = shear stress at a section of a beam or slab.

t_b = permissible stress in helical reinforcement.

t_w = permissible tensile stress in the shear reinforcement.

u = ultimate concrete stress = cube crushing strength.

u_p = u for preliminary test.

u_w = u for works test.

W = total load on beam or slab.

W_l = load on long span beam supporting a slab spanning in two directions.

W_s = load on short span beam supporting a slab spanning in two directions.

w = total load per unit area of slab or per unit length of beam.

Z = bending moment coefficient (suffixes as required).

Z_x and Z_y = bending moment coefficients for the short and long spans respectively, for slabs spanning in two directions and simply supported on four sides.

Z'_x and Z'_y = bending moment coefficients for the short and long spans respectively for rectangular panels supported on four sides and with provision for torsion at corners.

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